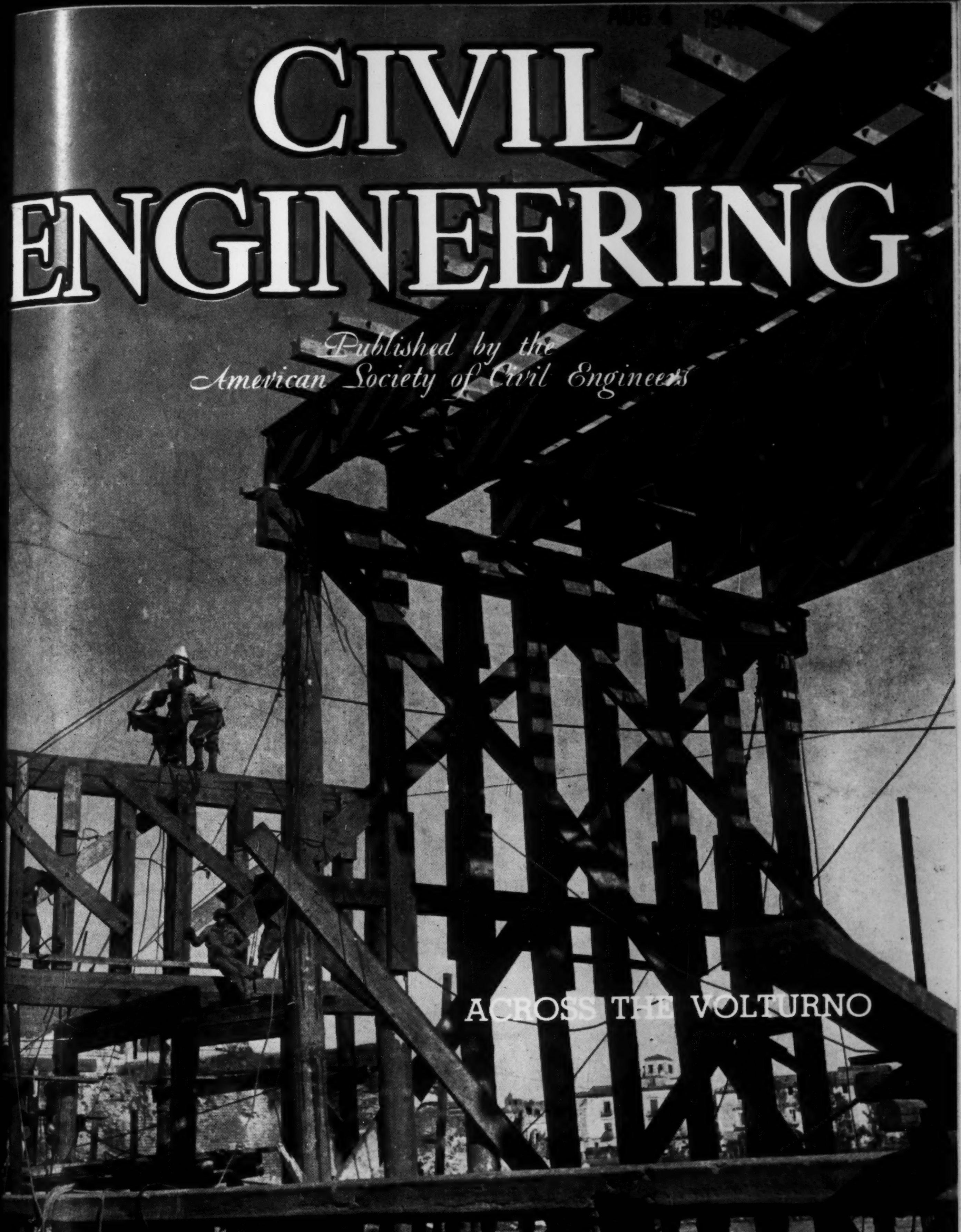
AUG 4 1944

CIVIL ENGINEERING

Published by the
American Society of Civil Engineers



ACROSS THE VOLTURNO

Volume 14 Number 8
AUGUST 1944

FOR THE CARGO TO B
LOADED TOMORRO

In order to establish a world-wide bala
of economic security and stabilizat
America must help prepare the way
foreign governments to open their do
to a universal interchange of produ
and ideas.

Today is the time for us to look beyo
the horizon and lay solid plans for bu
ing ports, piers, docks, jetties, harbor
provements and other post-war devel
ments. These are the constructions wh
will need firm foundations for sponsor
new industries... new communities
new friendships in the world of tomorr

The ability of the Raymond organi
tion to fulfill completely the obligatio
assumes is a recognized fact the wo
over. For 47 years owners, architects a
engineers who want difficult work do
exactly right have formed the habi
seeking Raymond again and again. Wr
wire, cable or phone for a compet
Raymond engineer to discuss your p
ject with you.

RAYMON
CONCRETE PILE COMPA

Branch Offices in Principal Cities

140 CEDAR STREET · NEW YORK 6, N

SCOPE OF RAYMOND'S ACTIVITIES includes every recog
type of pile foundation – concrete, composite, precast, steel,
and wood. Also caissons, construction involving shore prote
ship building facilities, harbor and river improvements
borings for soil investigation.

Among Our Writers

Donald R. Warren (Calif. Inst. of Tech., B.S.) is President of Donald R. Warren Co., which has handled the design of $300,000,000 worth of defense and war plants, the largest being the Kaiser Co.'s iron and steel mill at Fontana, Calif.

Frederick J. Converse (U. of Rochester, B.S. in M.E. '14) has been on the teaching and research staff of the Calif. Inst. of Technology since 1920, and is now Associate Prof. of Civil Engineering. His consulting practice has included much work on foundations.

Charles T. Leeds (U.S. Military Academy, '03; M.I.T., '06) served as an officer of the Corps of Engineers prior to and during World War I, with varied experience in river and harbor improvement and fortification construction. He is a member of Leeds, Hill, Barnard & Jewett.

Ralf R. Woolley (U. of Utah, B.S. Mining Eng. '08, C.E. '20) has spent 35 years on investigation and engineering research covering the water resources of the intermountain region in their relation to the overall development of the West. Special emphasis has been given to water power, irrigation, and other uses of water.

H. G. Acres (U. of Toronto, Applied Science deg. '03, Mech. and Elec. Eng. deg. '16) was chief hydraulic engineer for the Hydroelectric Power Commission from 1911 to 1924, and since then has been in private practice. He has been in charge of design and construction of many important power projects, transmission lines, and storage dams.

Edgar A. B. Spencer (Columbia College '13; Columbia U. Law School '15) is a member of the firm of Spencer and Iserman, special counsel for municipalities, and attorneys for firms in the construction trades.

Robert E. Dempsey (Colgate College '24; New York U. Law School '28) is Town Counsel for the Town of Cortlandt, Westchester County, New York. He also has represented contractors and municipal corporations in the State of New York.

Frederick B. Duis spent his entire professional career with the U.S. Engineer Department—the first 10 years in the Chicago District and the remaining 36 years on design, construction, maintenance, and operation of locks and dams, mainly in the Ohio River and its canalized tributaries. He was the principal civilian engineer from 1920 to his retirement in 1941.

C. N. Crocker (U. of Georgia, B.S. C.E. '15) has been with the Bridge Dept. of the Georgia State Highway Dept. since 1920, as Bridge Engineer since 1933. He is on the bridge committees of the Amer. Assoc. of State Highway Officials and Southeastern Assoc. of State Highway Officials.

George R. Rich (Worcester Poly. Inst., B.S. in C.E., '19) has had experience in major hydraulic, steam, and industrial engineering with Stone and Webster, McClellan and Junkersfeld, Jackson and Moreland, and the U.S. Engineer Department. He became head mechanical engineer of the TVA in 1937 and is now chief design engineer.

Robert Zaborowski (U. of Ill. '38, M.S. in C.E., '40) engineer for the Applied Physics Lab. of the Johns Hopkins Univ. engaged in work for the O.S.R.D. at Silver Spring, Md., has been on leave-of-absence from Roberts and Schaefer Co., Chicago, Ill. since May '44. Since 1941 he has been structural engineer in charge of the design of thin shells for Roberts and Schaefer Co.

Otto Gruenwald (Inst. of Technology, Vienna, '20) worked as a structural engineer with contractors and Austrian engineering firms, coming to the U.S. in 1938, where he first worked as a structural designer with the W. F. Jackson Company in Washington, D. C. In 1941, he joined the staff of Roberts and Schaefer Company.

VOLUME 14 NUMBER 8

August 1944

COPYRIGHT, 1944, BY THE
AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in U. S. A.

Entered as second-class matter September 23, 1930, at the Post Office at Easton, Pa., under the Act of August 24, 1912, and accepted for mailing at special rate of postage provided for in Section 1102, Act of October 3, 1917, authorized on July 5, 1918.

CIVIL ENGINEERING

Published Monthly by the
AMERICAN SOCIETY OF CIVIL ENGINEERS
(Founded November 5, 1852)

PUBLICATION OFFICE: 20TH AND NORTHAMPTON STREETS, EASTON, PA.
EDITORIAL AND ADVERTISING DEPARTMENTS:
33 WEST 39TH STREET, NEW YORK

This Issue Contains

COVER—Across the Volturno (*See article, page 329*)

NOVEL CONSTRUCTION PLAN FOR GRAVING DOCK SUGGESTED BY SOIL STUDIES 323
Donald R. Warren, Frederick J. Converse, and Charles T. Leeds

ITALIAN BRIDGE RECONSTRUCTED BY CORPS OF ENGINEERS 329
A. C. Kraft

AN ENGINEER LOOKS AT GREAT SALT LAKE 332
Ralf R. Woolley

THE SHIPSHAW HYDROELECTRIC PROJECT, CANADA II. Design for Development of a Million Horsepower 335
H. G. Acres

MUNICIPAL POSTWAR PLANNING 339
Edgar A. B. Spencer and Robert E. Dempsey

SURVEY OF NAVIGATION ON THE OHIO RIVER 343
Frederick B. Duis

COASTAL HIGHWAY SPANS CROSS GEORGIA DELTA 347
C. N. Crocker

BASIC HYDRAULICS OF WATER STORAGE PROJECTS. I. Project Layout and Lock Filling Systems 351
George R. Rich

SEAPLANE HANGARS OF CONCRETE 355
Robert Zaborowski and Otto Gruenwald

ENGINEERS' NOTEBOOK
Shear Failures Under Footings 360
Paul Andersen

Restricted Conditions in Direct Design of Indeterminate Trusses 361
A. G. Strandhagen

OUR READERS SAY 362
SOCIETY AFFAIRS 364
ITEMS OF INTEREST 372
NEWS OF ENGINEERS 373
DECEASED 374
CHANGES IN MEMBERSHIP GRADES 14
APPLICATIONS FOR ADMISSION AND TRANSFER 18
ENGINEERING SOCIETIES' PERSONNEL SERVICE 22
CURRENT PERIODICAL LITERATURE 24, 26, 28
EQUIPMENT, MATERIALS AND METHODS 30, 33, 34, 35, 36, 38
INDEX TO ADVERTISERS 40

The Society is not responsible for any statements made or opinions expressed in its publications.

Reprints from this publication may be made on condition that full credit be given CIVIL ENGINEERING *and the author, and that date of publication be stated.*

SUBSCRIPTION RATES

Price 50 cents a copy; $5.00 a year in advance; $4.00 a year to members and to libraries; and $2.50 a year to members of Student Chapters. Canadian postage 75 cents and foreign postage $1.50 additional.

Member Audit Bureau of Circulations

7 reasons... count 'em
. . . FOR USING ALUMINUM PAINT
HIGH REFLECTIVITY Makes bridges and guardrails easier to see
HIGH DURABILITY Cuts maintenance costs
RESISTANCE TO FUMES Doesn't readily discolor
HIGH HIDING POWER Hides with one coat
RESISTANCE TO HEAT "Stays put" on - hot surfaces
RESISTANCE TO MOISTURE Protects against rusting
ATTRACTIVE APPEARANCE Enhances beauty of structures

It's a mistake to judge paints on just one or two points, such as cost or appearance. Paints should be compared on all their qualities.

In comparing aluminum paint with any other there are at least seven advantages to weigh. To skip over even one is a mistake because, in varying degrees, each is important to you on most jobs. And because aluminum paint excels in so many points of comparison, you will find it hard to equal when you sum up all its values

You have been denied this fine paint for several years but now you get it again for use where you have used it before on bridges, highway guardrails and steel transmission towers. For latest information on other regulations covering aluminum paint, consult your paint supplier or WPB office. ALUMINUM COMPANY OF AMERICA, 1918 Gulf Bldg., Pittsburgh 19. Pa.

ALCOA ALUMINUM

ALBRON *Pigments for*

ALUMINUM PAINT

ALCOA ALBRON PIGMENTS ARE MADE UNDER PATENTS OF METALS DISINTEGRATING CO., INC.

MALCOLM PIRNIE
President

GEORGE T. SEABURY
Secretary

YDNEY WILMOT
Editor in Chief and
Manager of Publications

DON P. REYNOLDS
Assistant Editor

CIVIL
ENGINEERING

COMMITTEE ON PUBLICATIONS
SCOTT B. LILLY
Chairman
N. W. DOUGHERTY
S. C. HOLLISTER
FRED C. SCOBEY
WILBUR M. WILSON
W. L. GLENZING
Advertising Manager

VOLUME 14 AUGUST 1944 NUMBER 8

Novel Construction Plan for Graving Dock Suggested by Soil Studies

ALTHOUGH located essentially on the open seacoast, the West Coast graving dock here described was constructed in the dry by means of controlled pumping. Thus the expense and difficulty of underwater work were avoided. Success for the method was dependent upon the peculiar geology of the chosen site. The papers of this symposium, which were presented before the Soil Mechanics and Foundations Division, at the Society's Los Angeles Convention, discuss the unusual construction procedure used, the extensive soil testing required, and the expedients necessary to protect adjacent properties.

Construction Procedure

By DONALD R. WARREN, M. AM. SOC. C.E.
CONSULTING ENGINEER, LOS ANGELES, CALIF.

DESIGNED to handle the largest battleships afloat, the graving dock under discussion has sills at the entrance 45 ft below mean lower low water. The base of the foundation is approximately −60 ft. The fleet operating base, of which this graving dock is one of the units, was largely made by dredging the offshore channel and harbor area and using the materials to reclaim an open beach, so that, although it now appears to be entirely inshore, two-thirds of the graving dock extends beyond the original shoreline.

In reality, the graving dock (Fig. 1) was largely constructed in the open ocean, the depth of water at the outer end being originally 18 to 20 ft below mean lower low water. The average daily tide is approximately 5 ft, with a maximum tidal range of $10^{1}/_{2}$ ft. For some winds, the direct impact of the ocean waves is dampened by a breakwater offshore, but in one direction the reach is practically unbroken.

The dock was designed by the U.S. Navy's Bureau of Yards and Docks, for construction by underwater methods. It was not until the foundation investigation and laboratory analyses had been completed that any thought was given to constructing the dock in the dry. Studies of the grain size, arrangement of soil particles, sequence of strata, and preliminary estimates, definitely indicated that this could be done, and if done, would save both time and money.

As this was somewhat of a departure from the normal type of construction for a structure of this size, it was felt that the laboratory studies should be supplemented by a large-scale field test, in order to demonstrate the feasibility of this method of construction and obtain permeability coefficients and other data for design purposes. The test was successfully carried out under the direct supervision of the Bureau of Yards and Docks engineers.

The dock was excavated with a hydraulic suction dredge, and a rock sea wall was placed about the ocean end of the site to protect it from the waves. Immediately behind this sea wall a sand fill was constructed. Steel sheet-piles were driven through the sand fill and into the watertight silty loam below to form a wall impervious to the ocean water.

Laboratory permeability tests and field pumping tests indicated that to prevent the ground water from rising into the dock area, 36,000 gal of water per minute must be handled. In order to provide against uncertainties, a factor of safety of 2 was used in selecting the pumping capacity of the plant. Thirty-six 2,000-gal deep-well pumps were installed. These penetrated the gravel strata below the first layer of silty loam underlying the dock area, and some were driven well below the second layer of dense silty loam some 100 ft below mean lower low water. To prevent construction difficulties caused by the inflow of water through the slopes and percolation through the bottom, the water was lowered gradually, and after five weeks the graving dock was unwatered. The flow of water into the dock during this period diminished from 36,000 gal per min to 28,000 gals.

WELL-POINTS ALSO USED

In addition to 36 deep-well pumps, three rows of well-points circled the construction slopes to prevent surface seepage into the dock area. A total of 1,600 well-points, on approximately 4-ft centers, with 24-ft risers, were used for these protective measures. To encourage the flow of water to the well-points, each of the inlets was impacted in pea gravel. Continuous pumping kept the water table well below the excavation slope. The maximum yield from these well-points was 2,000 gal per min. When pumping started, the salinity of the water from the deep-well pumps was approximately one-third that of sea water. This percentage gradually diminished, but at all times the water from the substrata was definitely saline.

When the excavation had been dewatered and trimmed to the finish lines, a blanket of gravel approximately

VIEW OF DOCK SITE DURING CONSTRUCTION
Timber Construction Trestle Surrounds Excavation

$1^1/_2$ ft thick was placed over the entire base. On this material the base of the dock was poured in alternate transverse blocks, 18 ft wide and 186 ft long, each containing about 2,300 cu yd of concrete. When completed the dock contained 200,000 cu yd of concrete, all placed by buckets from overhead traveling cableways. Cableways were originally chosen because they were particularly adapted to tremie pours, but they proved very satisfactory for the method finally used.

STRAINS DEVELOPED DURING CONSTRUCTION

The construction methods did not develop strains in the dock in the same manner that the designer calculated, for the heavy walls placed on the outer ends of the base strips caused greater moment in the center of the slab than the completed design called for. The dock was designed to act as a rigid frame, and it was contemplated that the back fill on the exterior walls would provide the required fixity. During construction and before backfilling, a crack lengthwise of the dock opened to a width of about 3/ in.; however, this was not serious. The crack was grouted, and when the backfills were placed and the loads distributed in accordance with the original design, all was satisfactory.

The deep-well submersible pumps and motors used in the construction process were later employed for the dock pumping plant instead of the few large pumps contemplated in the original plan. These pumps, in addition to being a cheaper installation, had the advantage of providing a more uniform withdrawal of water from the entire dock and of reducing cross currents. They were also most dependable from the point of view of maintenance, for in case of an emergency, individual pumps could be overhauled without impairing the utility of the dock.

Soil Test Procedures

By FREDERICK J. CONVERSE, M. AM. SOC. C.E.

CONSULTING ENGINEER, LOS ANGELES, CALIF.

PROBLEMS of soil mechanics involved in the construction of a graving dock fall into two categories—those pertaining to the design of the structure and those peculiar to construction. Design requirements involve the determination of the bearing capacity of the soil beneath the slab and abutments, the establishment of a soil constant or ratio of load to deformation of the soil, the bearing capacity and resistance to uplift of piles, and the lateral earth pressure against the sides of the dock. Construction problems require information regarding the character of the soil to be excavated, safe dredge slopes, construction of a cofferdam, and determination of the most feasible method of construction, such as unwatering the site in order to construct the graving dock in the dry, tremie pouring, or the sinking of a caisson.

The first step was to acquire a detailed knowledge of the soil composing the site, not only as to character and classification, but also as to structural properties. These data were obtained by means of 20 borings sunk to a depth of 150 ft below the surface. Samples of the soil were taken from the borings at frequent intervals and tested in the laboratory. A careful geological investigation of the entire area surrounding the site was also made, and proved of exceptional value in interpreting the boring data and in deciding certain features of construction.

SAMPLING PROCEDURE

Some of the details of the procedure used to obtain soil samples will be of interest. About half of the 2 borings made were located on dry land and the remainder extended through the surf area into gradually deepening water to a maximum depth of about 20 ft below mean lower low water. All holes were sunk by Gow rigs, those over water from platforms supported by a cluster of four piles. A 6-in. casing was first sunk by jetting and driving. First a length of casing was set in place and driven into the soil with a heavy hammer. A $2^1/_2$-in jetting pipe with hose connections at the top was then

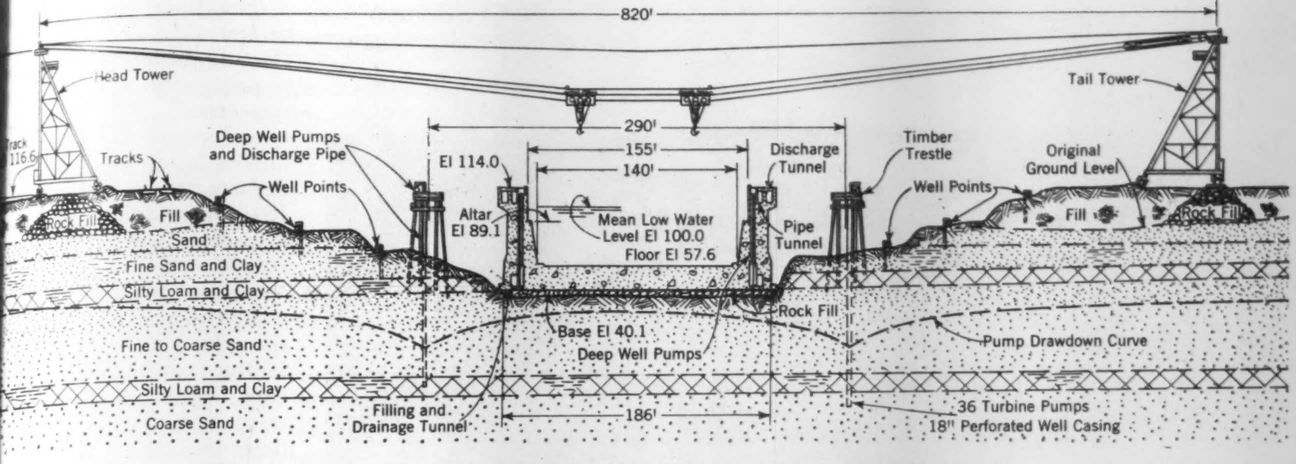


Fig. 1. Section Through the Dock Site Showing Construction Procedure

owered into the casing and water circulated down the et pipe and up to the top between the jet pipe and the casing.

The jet operated just below the bottom of the casing and the circulating water carried the loosened soil to the surface. This procedure worked smoothly and rapidly to a depth of 50 or 60 ft. At this depth, a stratum of very fine sand was encountered which flowed into the casing to a depth of several feet when jetting was stopped, and made sampling impossible. This difficulty was overcome by circulating mud instead of water.

Where gravel was encountered a chopping bit was fastened to the end of the jet pipe, and the gravel was either broken up and brought to the surface with the circulating mud, or driven away from the bottom of the casing into the sides of the hole. When this casing reached a depth where driving was difficult, a smaller casing was placed inside it and sunk to the required depth.

Soil samples were taken at each change in the character of the material, and at frequent intervals regardless of changes. Changes in character of the material were easily detected in the upper 50 ft by watching the wash water. When the depth became too great for this procedure, or when greater accuracy was necessary, continuous cores were taken. Samples for laboratory use were 2.41 in. in diameter, and were obtained by one of two methods, depending on the character of the soil. In material of relatively low density, such as the upper 60 ft of silt, clay, and sand, a spoon of the split-barrel type was used. The split barrel was 12 in. long and was lined with thin brass rings, each 1 in. long. The cutting tip was attached to one end of the split barrel and a 12-in. waste barrel to the other end.

A driving head, containing two ball valves in series, was attached to the upper end of the waste barrel. The cutting point was 2.4 in. in diameter on the inside, 0.01 in. less than the inside diameter of the liner rings. The cutting point tapered outward from a fairly sharp edge to a diameter of 3.5 in. in a length of $1^3/_4$ in. The long taper reduced distortion of the sample due to compaction in driving, but the high ratio of the cross-sectional area of the tube to the area of the sample of 1.12 prevented the successful use of the spoon in dense material. In fine sand and soft mud, a core catcher was used between the cutting tip and the split barrel. This device consisted of eight triangular brass leaves which folded down as the spoon was pulled, and gave nearly complete closure of the opening. The spoon was driven by oil-well drill jars and a heavy follower, operated by a rope around the cat-head of the drilling motor. The usual sample length was from 18 to 24 in., the central portion in the split barrel being preserved for laboratory tests.

Sampling in dense sand and tough clay was done with a Shelby steel tube with the same inside diameter as that of the split-tube spoon. The tube was approximately 30 in. long and was fitted with a driving head similar to that of the split sampler. The low ratio of the cross-sectional area of the tube to the area of the sample of 0.10, made this spoon easy to drive in all materials except gravel, which damaged the cutting edge of the tube. Only the center portion of the sample was used for test, experience indicating that the upper portion was frequently disturbed by the jetting and cleaning, and the lower portion by compacting near the cutting edge.

When using the split-tube spoon, the split tube was opened as soon as a sample was brought to the surface and two 6-in. sections, still confined in the brass liner rings, were transferred to close-fitting metal containers and sealed. In the case of the Shelby tube spoons, the containers were lined with the brass rings and the sample forced out of the tube directly into the containers.

In locations where exact depths to changes in strata were desired, a tube $1^1/_4$ in. in diameter was driven past the point where the laboratory sample was taken. The disturbed samples thus obtained were used as an aid in keeping an accurate field log and were not sent to the laboratory.

LABORATORY TESTS

Shear tests were made on practically every sample. The shearing resistance was usually determined at a normal load equal to the calculated buoyant weight of the soil above the point of sampling. Consolidation tests were made on representative samples. Owing to the character of the soil below the graving dock slab and foundations, large numbers of consolidation tests were not required.

Equipment for making both shear and consolidation tests was designed to receive the sample still confined within the brass liner rings in which it was taken in the field. This method has the advantage of preventing lateral expansion of samples taken from great depths and of minimizing the possibility of disturbances due to handling. All samples were broken in two after testing and examined for evidences of side drag, or distortion due to the tendency of some soils to cling to the tube and bulge in the center. Remarkably little disturbance of this nature was found. Whenever examination indicated

AFTER WATER LEVEL WAS LOWERED BY PUMPING STABLE MATERIAL WAS ENCOUNTERED

that serious disturbance had taken place, samples were cut down to a smaller size and check tests were run. Complete mechanical analyses, tests for specific gravity and the atterberg limit, were run on all samples from several test holes. The classification of other samples was determined by visual comparison.

FIELD LOAD TEST

In order to supplement the laboratory determinations of the bearing values and elastic properties of the soil, a field load test was conducted. To accomplish this, a steel casing 3 ft in diameter was first sunk by jetting and jacking to a depth of 62.5 ft. This placed the bottom of the casing in fine sand, and it was feared that some disturbance would occur in spite of all the care that could be exercised in bailing and cleaning. For this reason a circular test plate 3.83 sq ft in area, was welded to the end of a 10-in. pipe and worked through the sand to a depth of 6 in. below the point where it first landed.

Load was then applied by means of a hydraulic jack, the reaction being furnished by a double I-beam welded to the outer casing. The test loads were measured on a special gage located between the jack and the reaction beams. Settlements of the plate were obtained by attaching a wire directly to the plate and bringing it to the surface, where its movements were measured by a dial gage reading to 0.001 in. Check readings were made on the top of the pipe by stretching a wire between two posts. Further checks were also made at intervals of eight hours by direct level observations. Loads were applied in increments of 0.66 tons per sq ft, each load being left in place until the rate of settlement was less than 0.001 in. per hour. In order to determine the elastic action of the soil, rebounds were taken at several loads.

In order to calculate the stresses in the bottom slab of the graving dock, considering the slab as an elastic beam in an elastic foundation, the designer required the elastic constant of the foundation. This can be determined either from the field load test or from the laboratory consolidation tests. Both methods give practically the same results.

The high bearing capaci of the sand and gravel belo the foundation level made pil unnecessary as far as load w concerned. Piles would, ho ever, reduce the deformatio of the slab to less than half t deformations without pile In uplift, the wood piles showe a resistance approximate equal to the buoyant weight a column of soil 3 ft square a a depth equal to the length the pile. This was not sati factory and indicated th some other type of pile anchor should be used. Th designing engineers chose t use mass concrete.

A study of the geologic sections constructed from th boring logs (Fig. 2) shows non-uniform strata of fine mi caceous sand, silty-loam, clay loam and clay, extending dow to an average elevation −60 ft, but varying from −4 to −75 ft. This material was low in density and als in general, in permeability. Clean sand and grave underlie this stratum and extend to El. −112. Belo the sand lies a moderately dense and impermeable blanke of silty-loam and clay from 10 to 20 ft thick, followe by coarse sand and gravel to the depth investigate Near-by oil-well logs indicate that this permeable ma terial extends down to about El. −200 and is followed b a very thick stratum of dense clay. From the genera geology of the region, it is known that both the uppe and lower sand and gravel strata carry a flow of fres water from the land to the ocean.

For purposes of discussion, the upper, relatively im permeable stratum, from the surface to −60 (+ or − will be designated as "A," the upper sand and grave layer as "B," the silty loam blanket at El. −112 a "C," and the lower sand and gravel stratum as "D. Since the bottom of the graving dock penetrated int stratum "B," any attempt at unwatering would involv taking care of the entire flow of water in this stratum both from the land and the ocean. Since stratum "C acted as an impermeable blanket between "B" an "D," a differential pressure of about 75 ft of water woul exist between "B" and "D" when pressure in "B" wa lowered the required amount. It was feared that thi might be enough to cause a blowout through the blanke and it was therefore deemed advisable to pump fro "D" as well as from "B."

In order to obtain information relative to the feasibilit of dewatering the site, an experimental pit was sunk The top of the pit was roughly 60 ft square. Well-points were put down as excavation progressed. Ex cavation of the pit was carried to El. −49, but at thi depth a considerable flow of sand occurred below th sheathing, and it was not considered worth while t carry the excavation deeper.

Four deep-well pumps were installed at the corners o the pit, extending into zone "B." First, 14-in. casing were put down to the required depth and sealed of The casings were perforated at the lower ends for distances varying from 9 to 35 ft, the elevations of the lower ends of the casings varying from −75 to −93 ft The deep-well turbine pumps, installed within the casngs, developed a total discharge from all installations f 2,900 gal per min. Six observation wells were sunk n an east-west line through the pit.

Approximately six weeks after the first deep-well pump tarted working, and two weeks after the fourth well was ut in operation, the drawdown line, as determined from he observation wells, was considered stable. The slope f this line at any point was taken as the hydraulic gradient at that point, and from this value and the known ischarge of 3,000 gal per min, the coefficient of perneability, k, was calculated. Since observation of the ow from the well-points indicated that very little water as being drawn from zone "A," the thickness of zone "B" was used in determining the area through which ater was flowing. The value of k determined by this neans was 3.03. The maximum hydraulic gradient of he drawdown curve was 0.30.

In order to determine the total pumping capacity reuired to dewater the dock site, the total thickness of oth zones "B" and "D" was used and taken as 130 ft, nd the perimeter of the site was taken as 2,600 ft. The otal pump capacity required was calculated to be 7,800 gal per min. Pumping this quantity of water as considered entirely feasible, and plans were made for ewatering the site.

In the dewatering operations, 36 deep-well pumps ated at 2,000 gal per min each were installed, arranged o as to entirely surround the dock site outside of the inished structure. The pumps were operated from a latform carried on long piles. The wells were contructed by first sinking a 30-in. conductor to El. −55. Within the conductor an 18-in. casing was sunk well nto zone "D" at various maximum depths down to El. −140. These casings were perforated throughout most of their contact with the sand and gravel strata. The space between the 30-in. conductor and the 18-in. asing was packed with approximately 15 tons of pea ravel.

Forty days after work was begun on the deep-well umps, and 25 days after all 36 pumps were in operation, the dock was completely dewatered to El. −62. The average pumping rate during this period was approximately 36,000 gal per min. A continuous pumping rate of 20,000 to 25,000 gal per min was required to maintain the drawdown. As the water in the dredged pool was lowered, some seepage came through the sides. This was handled by well-points, put down in three rows, from which a maximum of about 2,000 gal per min was pumped.

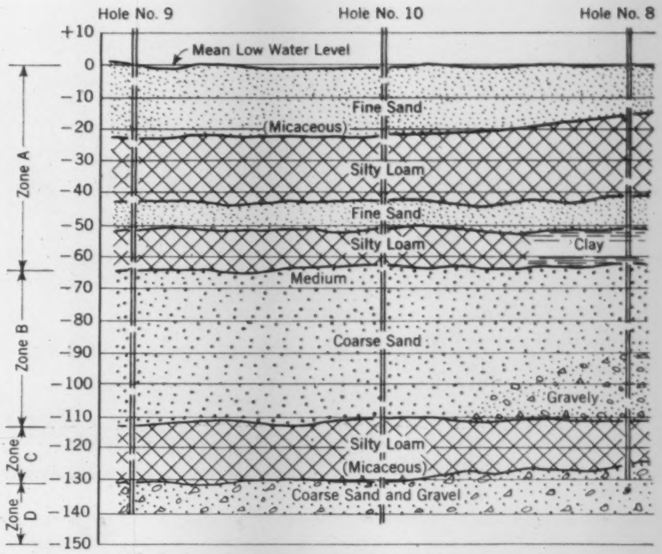


Fig. 2. Boring Log of Dock Site, Typical Section

Pumping was continued for about eight months, until the structure was completed. The rate of pumping was then gradually cut and the water level allowed to return to normal. As a construction measure the procedure was a distinct success, resulting in a very substantial saving in both time and money as compared to other methods of construction.

Protection of Adjacent Properties

By Charles T. Leeds, M. Am. Soc. C.E.

Consulting Engineer, Los Angeles, Calif.

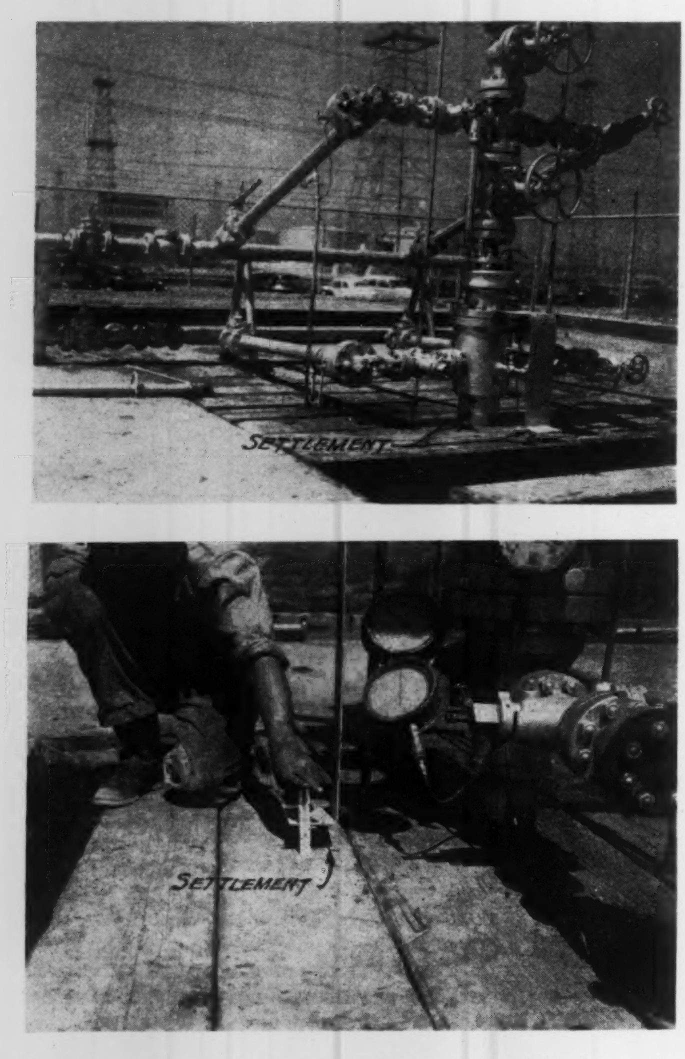
LOCATION of the graving dock in a highly developed area made necessary extensive studies to determine the proper unwatering method. Apparently, the hazards to surrounding property which the pumping procedure entailed were not at first fully recognized. Later, however, the expense of precautionary measures was balanced against the estimated cost of damages.

There are numerous oil wells in this locality and when, as a result of the pumping, many of the concrete slabs surrounding the tops of these wells broke loose from the well casings and moved downward as much as 3 or 4 in., fears naturally arose lest increasing settlement should cause difficulty of operation and possibly affect safety.

A greater cause of concern than the oil wells was the presence of a major-sized industrial plant, vital to the community, consisting of three large reinforced-concrete buildings containing operating machinery. The nearest building is some 1,500 ft from the shoreward end of the dry dock. This building, though constructed with a heavy concrete mat on piling, has attached to it various auxiliary structures which are not founded on piles. Therefore there was danger of uneven settlement, causing misalinement of the machinery or breaking of the tunnel and pipe-line connections between the main building and the auxiliary structures. Accordingly, when it was found, shortly after pumping started, that water levels in two wells close to the plant had dropped from +4 to −27 and −31 ft, respectively, and that the corner of the building nearest the dry dock had settled 1/4 in., this settlement doubling in the next two weeks, it was not surprising that officials became concerned.

SUBSIDENCE RECORDS KEPT

Careful level circuits were therefore run periodically to determine the change in elevation of critical points, care being taken to include certain reference points, the stability of which was clearly evident. Time-subsidence graphs of these points were then drawn and kept constantly up to date. While the subsidence appeared to be definitely related to the rate of pumping, the rate of subsidence was far from uniform throughout the area. The variation seemed to bear a definite relation to the geological structure beneath, as would be ex-



SETTLEMENT DUE TO LOWERING OF WATER TABLE WAS APPARENT IN ADJACENT OIL FIELD

pected. As far as could be learned from the data and old maps available, it appeared to bear a direct relation to the ancient river channels which had been cut and refilled repeatedly in decades or centuries past.

The geological formation shoreward of the dry dock, in the area where the oil wells and industrial plant were located and where the settlement took place, is generally similar to that shown in Fig. 2. Actually it varies considerably owing to historic wanderings of the old river channels.

In the vicinity of the plant are five wells, three of which are only 35 ft deep, while the other two are 155 ft and 359 ft deep. The water levels in the first three were not affected by the dry-dock pumping, but the levels in the other two were, as has been stated. Also, in these deep wells, the water levels fluctuated directly with the tidal fluctuations in the ocean, although the changes of level in the wells were only a fraction of the tidal range, and there was no appreciable time lag behind the corresponding tidal fluctuations. Apparently, therefore, the well-level fluctuations were due to direct transmission of pressure from the tide.

Because of the concern felt as a result of this ground settlement, a consulting board was appointed. It was the opinion of this board that the settlements were clearly caused by the relief of artesian pressure in the water-bearing strata ("B" and "D," Fig. 2) and consequent reduction in the buoyancy of the soil, resulting in some elastic compression and probably some actual consolidation of the compressible strata. Confronted with the prospect of continued heavy pumping from the dry-dock site in order to lower the water level in it to −70 and maintain it there until the concrete base of the dock had been poured, and a succeeding pumping period of some months at a lesser rate, the board deemed it prudent to take remedial measures.

With the cooperation of the Naval Public Works Officer, a program of repressuring was decided on, according to which water would be pumped back into the strata that were being depleted by the dry-dock pumping. Another suggestion was to obstruct the permeable strata between the dry dock and the plant by injecting mud, cement grout, or chemicals, and then to repressure behind the barrier thus formed. This latter method was discarded as not feasible because of the thickness of water-bearing stratum "D".

As a first and test stage of the program, two 28-in. cased and gravel-packed wells were drilled to a depth of 154 ft, and four additional 4-in. cased observation holes were drilled to a depth of 150 ft. After thorough development by alternate pumping and refilling, each well yielded approximately 1,200 gal per min as a producing well and took approximately 600 gal per min when used for repressuring. An exact determination of the effect of repressuring was not possible, because of the impossibility of telling what the water levels in the various observation wells would have been without repressuring. However, by comparing the water levels in the various observation wells, as affected by the dry-dock pumping, both before and after repressuring, it was concluded that this method afforded a practicable means of restoring the water pressure near the plant.

SETTLEMENT PREDICTIONS LOWERED

The time-settlement curves now showed a distinct flattening and the settlement of the buildings had almost ceased. Meanwhile additional exploratory borings showed that there was considerably less compressible material underneath the plant buildings than elsewhere in the vicinity, so that settlement predictions could be decreased from those previously anticipated. Also, better progress than expected had been made on the dry dock, so that the rate of pumping there had passed its peak, and a material decrease in the pumping rate was in prospect.

The estimated cost of an adequate repressuring program was found to be greater than the cost of repairing such damage as might result from probable settlement during the ensuing year. Accordingly, it was decided to suspend the repressuring program but to leave all existing facilities in place until the dock pumping should be reduced sufficiently to cause a substantial rise in the ground-water level. As was expected, a major part of the subsidence has now been restored without any serious damage. This method of construction, when well planned and based on a thorough soil mechanics study, can effect major economies in cost and time of construction. However, in all cases careful consideration should be given to the probable effects on surrounding foundations.

Design and testing were under the supervision of the U.S. Navy, Bureau of Yards and Docks, with Messrs. Warren and Converse as consultants on foundations and soil investigations. The consulting board on settlement damage to adjacent property consisted of George L. Freeman, R. V. Labarre, and Charles T. Leeds, all Members Am. Soc. C.E.

Italian Bridge Reconstructed by Corps of Engineers

By A. C. Kraft, Lieutenant Colonel, Corps of Engineers, U.S. Army

On every fighting front engineers are working the miracles of construction that make the rapid advance of our forces possible. This story of the construction of the Capua Bridge across the Volturno River refers to just one of thousands of bridges replaced. The site of this accomplishment is well known as the scene of the bloodiest engagement of the Italian campaign. In his account Colonel Kraft depicts the skill and daring of the engineers.

ENEMY demolition of bridges in the early days of the Italian campaign was a triumph of efficiency. Narrow passes through the Apennine foothills of Campania provided German engineers with excellent opportunities to delay the progress of the Fifth Army in its march northward. Supply routes were difficult to establish, but much more difficult to maintain. Hurriedly constructed bypasses bulldozed across dry channels proved inadequate when the torrential rains came. The Bailey Bridge was widely used in the early days of the campaign, but the limited stock of parts for this remarkable structure was soon depleted. Thus it was necessary to build timber structures of a semipermanent nature to release tactical bridges for advance operations.

At the river Volturno, the scene of Hannibal's undoing, the enemy achieved the peak of destruction. Not only were all the bridges blown, but every single bay was reduced to rubble from shore to shore. Ponton bridges were launched by American and British engineers at Pte Annibale and at Capua, where the near-by ancient ruins of early Roman bridges looked not unlike the recent accomplishment of the German Pioneer Platoons. Advance units of the Fifth Army moved across the river driving the enemy northward several miles. The Army Engineer ordered immediate construction of two semipermanent bridges to replace the temporary crossings that were inadequate to handle the heavy guns and supplies vital to continued progress.

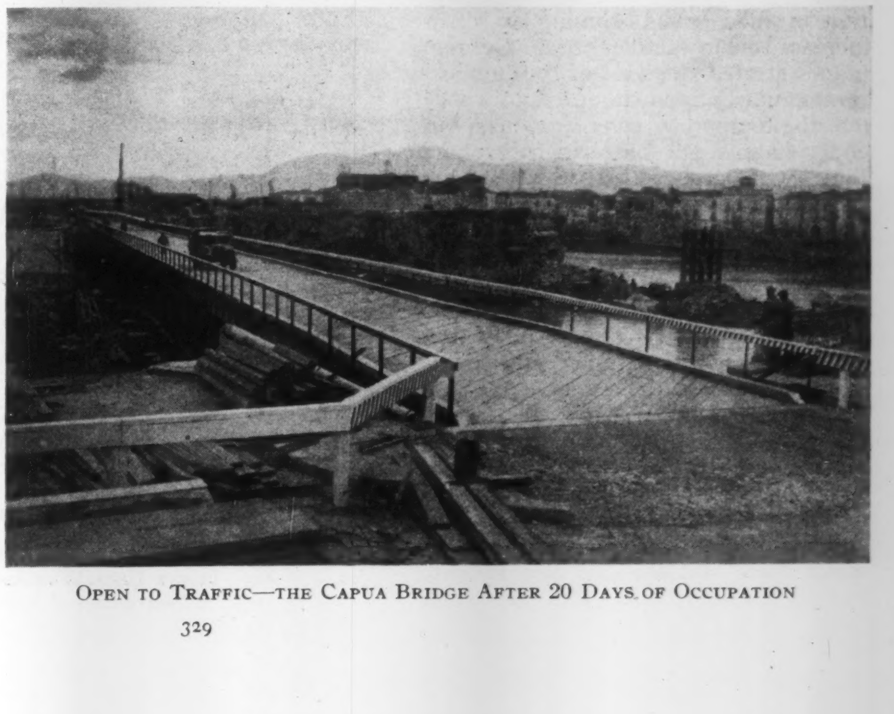
The 343d Engineer Regiment, veteran bridge builders of the African and Sicilian campaigns, was given the dual assignment, of which the bridge at Capua proved to be the major project. Col. Richard B. Dunbar, Assoc. M. Am. Soc. C.E., commanded this regiment. The 1st Battalion, under the command of Maj. Thomas E. Holt, was selected for construction of the bridge, and Capt. William E. Harrison, "A" Company Commander, was chosen to prepare the design.

Route 6—the famous Via Appia, the main highway to Rome—crosses the Volturno River at an elevation 32 ft above the normal water level. The south bank of the river consists of a vertical masonry retaining wall of equal height, and the north edge is an earth embankment approximately 25 ft high. A site for the new bridge was selected immediately upstream and parallel to the demolished structure in order to utilize the existing approaches with minor alterations. In full view of enemy patrols and machine gun fire, soundings were made under direction of M/Sgt. Charles W. Lipphardt, chief of party, and the location was probed with rods manipulated from one small boat and a hastily constructed raft. The rate of flow at that point was approximately 3 ft per sec, and many treacherous eddies added to the difficulties of the mission.

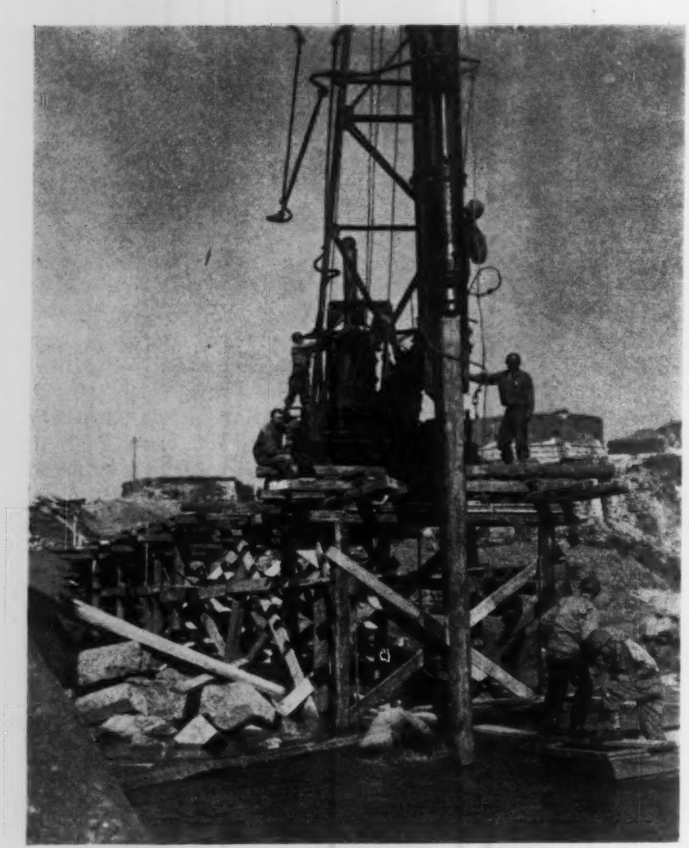
On the afternoon of the first day a preliminary survey was finished. Collected data revealed a stream 360 ft in width, with an average depth of 10 ft and a river bed of silt ranging from 6 to 20 ft on bed rock sloping from north to south. Since the existing approaches were to be used, the structure would be 32 ft high above the current water level and approximately 375 ft long. The order called for H-40 loads for two lanes, and H-70 loads for one.

MATERIALS AT HAND PRESSED INTO SERVICE

Unlike civilian practice, Army engineers in the field must design their work from a knowledge of construction materials available, and being in a foreign country does not make the situation easier. Experience in previous campaigns, however, paid dividends. At the Battalion CP, between the bursts of enemy artillery shells falling in the area, company officers in turn revealed the locations of various stock-piles of critical construction materials. It was learned that nearly enough timber piling for the job had been left in good condition by the Germans not far from the bridge site; a few power poles from wrecked civilian lines would make up the deficiency. Cap timbers and bent materials were known to be available in engineer depots at the rear. An old but salvageable pile-driving rig had been observed in a torpedo plant in Naples, and a reconnaissance party and interpreter left immediately to investigate. Steel girders were also in evidence near Naples and these, with some cutting and



Open to Traffic—the Capua Bridge After 20 Days of Occupation



WITH A SALVAGED RIG AND THE JERRY'S PILES A TEMPORARY TRESTLE WAS BEGUN

welding and the help of St. Elmo, could be made to do for bridge stringers. It was thought that decking beams could also be obtained there, so a second detail was dispatched to Naples. Miscellaneous steel and timber could be salvaged from the rubble of destroyed Capua. Thus a bridge plan was conceived.

By morning Captain Harrison presented sketch plans to Army Headquarters, and these were immediately approved by the Engineer. Daybreak of the third day found materials and equipment arriving at the job. Before noon the work had to be suspended as the concentration of men was beginning to draw increased enemy artillery fire. German planes strafed the lines of communication regularly and attempted to knock out the temporary river crossings, but effective ack-ack batteries prevented that. On October 21, 1943, after a lull of two days, conditions had improved sufficiently to permit operations to resume on a full scale.

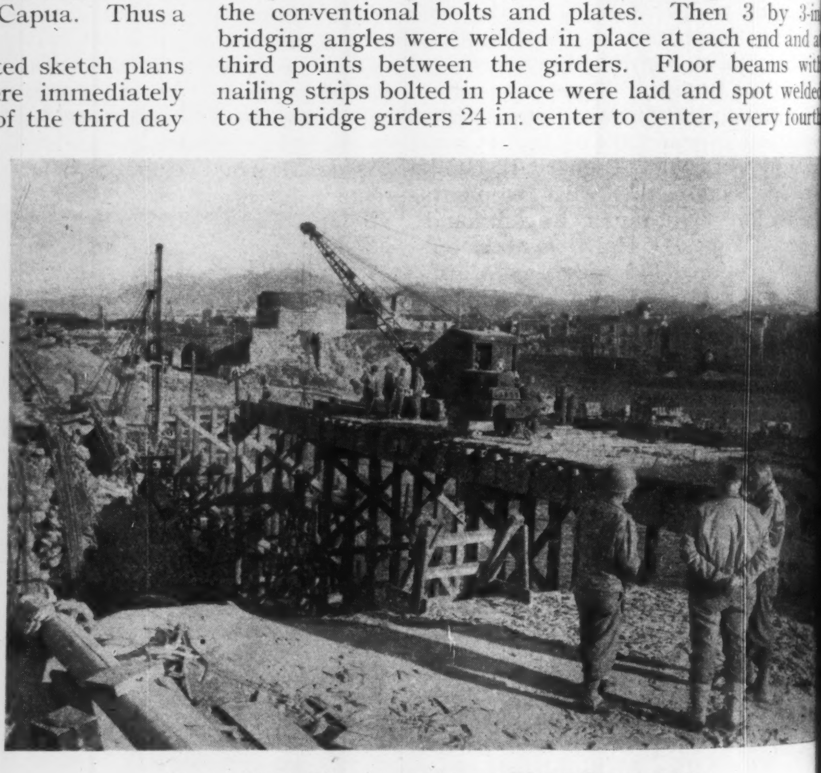
Construction was begun by ramping down the north bank of the river to an elevation 10 ft above the water. From that point a timber pile trestle was built across the river on the new bridge center line to carry the driving rig to the first set of permanent piling off the south bank. This causeway consisted of two telephone pole pile bents driven at 10-ft intervals, and capped and spanned with wooden stringers on which were laid the ties and rails to carry the pile driver. The pile driver was an Italian manufactured steam-driven rig fitted fo operation on a wide gage (8-ft 2-in.) railroad track. I was equipped with a 600-lb steam hammer and 480-f leads and could be moved forward and backward an turned through 360 deg on its own power.

PERMANENT PILES DRIVEN ON SIXTH DAY

On the sixth day the temporary trestle was complete and the first row of permanent piling was driven. Th original design called for a double row of timber piling but limited time and material resulted in the use of eigh 60-ft piles in line spaced at 3-ft centers. All bents drov about equally, the timber piles penetrating from 18 to 2 ft and requiring an average of one hour each. Piles wer trimmed and capped with 12 by 12-in. timbers and secured by steel side strips and drift pins. Horizonta braces and diagonal 3 by 12s were bolted on each row o piling. Timber bents constructed of eight 12-by-1 posts, caps, and sills were fabricated on shore and floate into a position from which they were raised in place b gin poles and winch line from a 4 by 6 on the south bank When the first bent was completed, the pile-driving ri started upon its return trip to the far bank, constructin each of the successive 10 bents in the manner describe Eight days were required for building the substructure.

On the south shore cutting and welding operation were being conducted on a 24-hour schedule. Prefabricated 30-in. steel girders found in a Naples plant had t be refabricated to suit the job. Eight mobile weldin units were in constant operation, tarpaulins obscuring th view from Jerry's nightly visits. For bolting nailin strips to the girders, 6-in. channels were flame cut t appropriate lengths. Random 2 and 3-in. deck plank were sorted and stockpiled for easy handling.

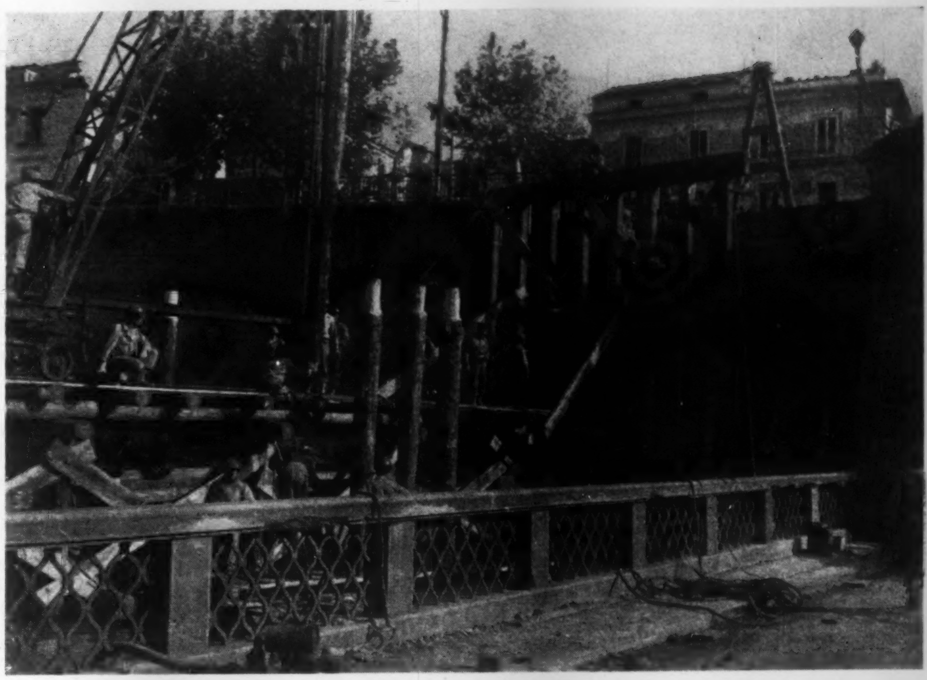
When the first bent was completed and secured, 3/4-in bearing plates were set and the six 30-in. steel girders wer placed, 4 ft center to center, by a half-yard crane rigge with the 35-ft component crane boom. A concret bridge seat had previously been built and provided wit the conventional bolts and plates. Then 3 by 3-in bridging angles were welded in place at each end and a third points between the girders. Floor beams wit nailing strips bolted in place were laid and spot welde to the bridge girders 24 in. center to center, every fourt



PLANK DECK WAS ADDED AS THE WORK PROGRESSED
Effectiveness of Enemy Demolition Shown at Left

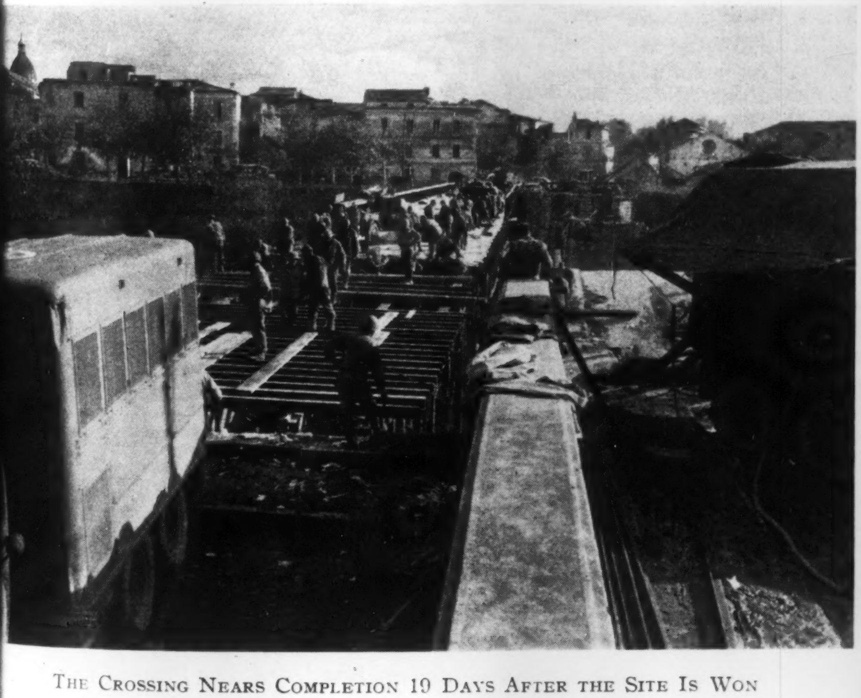
eam projecting 8 in. to provide a racket for the bridge railing. A in. native timber decking was ailed in place diagonally, followed y a 2-in. cross nailed at right ngles to the first. Timber wheel uards 10-in. square were bolted the deck providing a 23-ft clear oadway between.

No median-strip was used, thus llowing the center of the bridge to ccommodate heavier than H-40 ads without obstructions. A 2-in. railing consisted of 3-by-3 ngle iron posts and two wood rails 2 by 6-in. stock. Successive ays were constructed similarly, exept for the fact that 36-in. reabilitated open-web girders were ubstituted for the plate girders in e last four bays, since it was ecessary to use steel sections imediately available. The compleon of the superstructure required 2 days in all, daylight at that time sting about ten hours. For easons of security night operations ere obviously prohibited except the case of fabrication work.



First Bents of the New Bridge Take Form Beside Section of the Demolished Crossing

The north bridge abutment was constructed of native mber cribbing, slightly battered and filled with rubble om the demolished bridge. Approximately 200 ft of ad approach was constructed by caterpillar tractors nd with borrow from a convenient pit. Embankment opes were reinforced with riprap quarried locally. hen the river was fully spanned, diagonal cable was n longitudinally and transversely to provide continus bracing for the structure. Steel ship channels were olted from bent to bent approximately 10 ft above the ater. Bridge rails were painted alternate red and white ripes on the inside face, and the wheel guards were ainted white to assist in blackout driving.



The Crossing Nears Completion 10 Days After the Site Is Won

The Capua Bridge was officially opened to traffic at noon on November 9, twenty days after the actual starting date. The work was accomplished entirely by personnel of the 343d Engineer Regiment except for the operators of several supplementary welding units borrowed from other engineer battalions. The only piece of civilian equipment employed was the pile driver resurrected at Naples. Just short of 50,000 man-hours were expended in completion of the structure, a daily average of 250 men being employed for the 20-day period. A traffic count taken during the first two months revealed that an average of 10,000 vehicles passed over the bridge daily. At the end of the six-month period well over 1,000,000 vehicles had crossed the Capua Bridge into forward areas. Not only is the Capua Bridge the most traveled structure built by army engineers, but in size it outranks any similar project in the European Theater of Operations.

When the early spring rains started, every other bridge on the Volturno River either failed or was closed to traffic during the flood stage. In ten hours the stream rose 18 ft bringing down washed out bridges and debris in hazardous quantities, and failure of the other bridges added to the traffic burden of the Capua Bridge. Army commanders looked with grave concern at their last remaining contact with the front. But despite the incessant pounding of tanks, artillery, and heavily loaded lorries, and the lashing and battering of its high slender substructure by the flood-angry Volturno, the Capua Bridge stood, and stands today, a lasting tribute to the ingenuity and skill of the U.S. Army Engineers.

Southern Pacific R.R. Co.

"Going to Sea by Rail" Across Great Salt Lake

An Engineer Looks a Great Salt Lake

By Ralf R. Woolley, M. Am. Soc. C.E.

Senior Hydraulic Engineer, U.S. Geological Survey, Sal Lake City, Utah

GREAT Salt Lake was once nearly as large as Lake Michigan. At that stage (Fig. 1) it was 346 miles long, 145 miles wide, and 1,050 ft in extreme depth. The water then stood 850 ft deep on the Temple Block in Salt Lake City, 850 ft at Ogden, 650 ft at Provo, and 150 ft at Fort Douglas (Salt Lake City). If the water were to rise again to its old mark, the major part of Utah's population would be driven from their homes.

The most conspicuous traces that remain of this ancient lake are its shore lines. These are strikingly noticeable along the west slopes of the Wasatch Range and on the rocky islands of the present lake. They are also evident along the lower slopes of the mountains all about the old lake basin—not as a single line but a series of lines at many levels, indicating cycles of oscillations in the depth of the water.

As early as 1871, G. K. Gilbert ("The Geology of Portions of Nevada, Utah, California, and Arizona, 1871–1872: Geographical Explorations and Surveys West of the 100th Meridian," U.S. Engineer Office, p. 88, 1875) in his geologic studies in the Great Basin came to regard a series of lakes, of which he found the sediments and beaches, to be of the glacial epoch. The greatest of these was the forerunner of Great Salt Lake, for which he at that time proposed the name Bonneville. This was in honor of Capt. B. L. E. Bonneville, who gave an account of the lake published by Washington Irving about 1843 in "The Adventures of Captain Bonneville, U.S.A., in the Rocky Mountains and the Far West." Gilbert's subsequent studies and reports on Lake Bonneville finally culminated in Monograph No. 1, Lake Bonneville, published by the U.S. Geological Survey in 1890—a work that is highly regarded and is unique in its descriptive interpretation of the geologic evidences of this ancient lake.

Great Salt Lake today is less than one-tenth of its Bonneville size (Fig. 1) and the striking feature of its hydrography is its shallowness. It occupies no marked local depression but rests on the surface of a broad plain, only a slight oscillatory movement of which would decant the water into another area. In 1850 soundings taken by Stansbury indicated a maximum depth of 36 ft; now (April 1943), it is about 30.4 ft. Since records have been kept, its maximum depth was about $46^1/_2$ ft (in 1873), and its minimum about $28^1/_2$ ft (in 1940), a total fluctuation of 18 ft.

During the growth of Lake Bonneville, more moisture was precipitated in the basin than was carried away and thus the lake was produced. But the secular variation is in the direction of progressive shrinkage caused by a change in the balance between additive and subtractiv water supply factors—a movement which has been goin on in varying degree since the Bonneville epoch. It fundamental that if more moisture is contributed to th basin than is carried away the lake will grow, and con versely. Since man began to record lake elevation (about 1850) the trend has been downward at the aver age rate of about 1 ft in 15 years (Fig. 2).

Assuming that this secular movement toward desicca tion continues, it is obvious that ultimately a saline be will be all that is left of Great Salt Lake. It was a com mon belief about the turn of the century that the lak would reach this stage rather quickly, for it was falling a the rate of about half a foot a year. Saltair Pavilion, th unique picnic, dancing, and bathing resort that was buil in 1893 over 4 ft or more of water on the east edge of th lake directly west of Salt Lake City, had little or n water under it from the summer of 1902 to the end o 1906. The flat slope of the lake bed made it necessary fo bathers to walk out several hundred feet to reach wate deep enough for swimming.

In the public mind, the most popular reason for thi rapid falling of the lake was that "the climate is chang ing." To others, more technically minded, it seeme

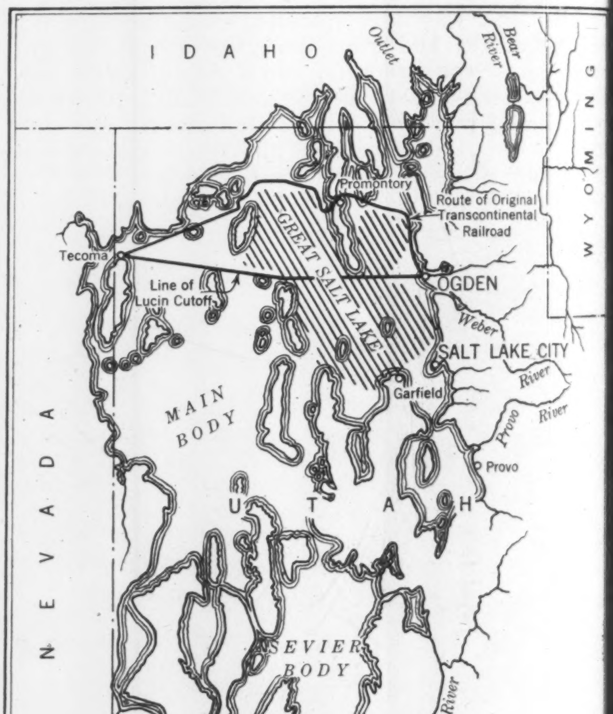


Fig. 1. Map of Ancient Lake Bonneville
Heavy Hatching Shows Area Covered by Present Great Salt La

gical that irrigation was an important cause. However, either of these theories accounts for the fact that the ke began to rise in the latter part of 1905 and coninued to do so for 19 years (Fig. 2). This rise totaled ft despite the fact that the total area irrigated from the reams that feed the lake increased from 535,000 acres 1902 to 849,000 acres in 1919. (Fourteenth Census S., 1920, Irrigation.)

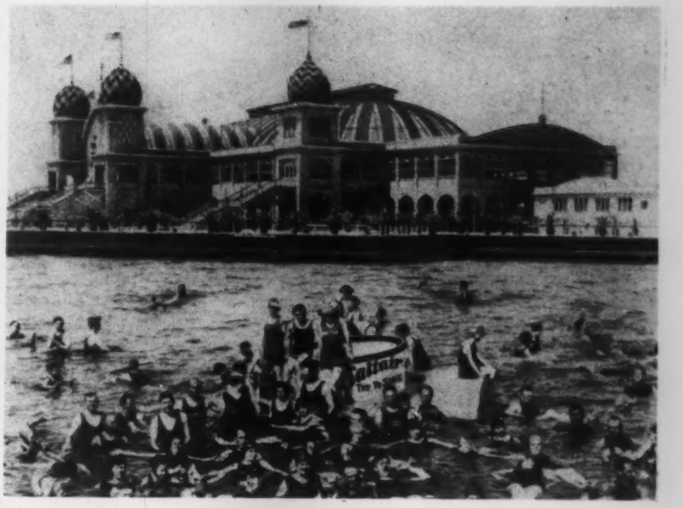
This anomaly then gave rise to the belief that water rought into the Great Salt Lake drainage basin from the trawberry Reservoir, in the Colorado River drainage asin, was materially contributing to the rise of the lake. he Strawberry Valley (Federal Reclamation) Project rigates some 60,000 acres south of Provo. It began peration in 1915, using an average of 64,000 acre-ft ach year, secured by trans-mountain diversion from the olorado River drainage basin. If 64,000 acre-ft of water ere poured directly into Great Salt Lake it would raise he present level approximately 1 in. Today the Saltair avilion is farther from the water's edge than it was in 906 and the lake surface is about 6 ft lower than when ater was first brought over from Strawberry Reservoir.

Great Salt Lake drains about 27,000 sq miles of mounainous highlands and low-lying plains in the northern art of Utah, and nearby Nevada, Idaho, and Wyoming. ractically the entire west half of this vast area is ocupied by the lake itself and the Great Salt Lake Desert, ith an annual precipitation of less than 5 in. The eastrn half is mountainous uplands draining through numerus small independent streams and three principal river ystems—the Bear, the Weber, and the Provo (Fig. 1). he latter empties into Utah Lake, which in turn disharges into Great Salt Lake through the Jordan River.

THREE MAIN TRIBUTARIES

These three principal streams rise in small lakes and prings in the glacial cirques on the west end of the inta Mountains less than 60 miles nearly due east from alt Lake City, in the Mirror Lake—Grandaddy Lake ecreational Area of the Wasatch National Forest. All ise within a small area of not more than 5 sq miles. he total length of the Bear River is approximately 300 iles, yet its mouth is only 75 miles from its headwaters. he Weber River's course is about 125 miles long, and its otal fall is 4,000 ft. The Provo River has a total length f about 70 miles.

Bear Lake at the north end of the Great Salt Lake asin, and Utah Lake at the south end, are fresh-water akes. Fluctuations in their levels are modified by dams and pumping plants designed to control the outflow for irrigation or power use.


Saltair Beach Co.

PRESENT SALTAIR PAVILION DATES FROM 1926
Original, Built in 1893, Burned in 1925

Great Salt Lake has no outlet. Its waters are salty and the amount of salts is slowly increasing as more dissolved minerals are brought in by streams, and left behind in the continuing process of evaporation. The percentage of salinity varies inversely with the changes in lake level or lake volume. For example, an analysis made in 1877 showed 13.79% of dissolved solids, and another made in 1904, when the lake level was much lower, showed 27.72% (see U.S.G.S. Bulletin 770, and Fig. 2). Since late in 1933 the lake has been saturated with respect to sodium salts.

The intense salinity of the lake precludes the existence of the ordinary aquatic forms of life. Considerable research study has been devoted to its microscopic organisms. Investigators have reported 9 or 10 species of animals and nearly a hundred species of plants. One of the animals is a minute brine shrimp, 3 are flies, and 5 or 6 are protozoans. Of the listed plants, 13 species are algae, 12 are bacteria, and 62 are diatoms. The salinity of the lake is from 4 to 10 times that of the ocean, yet the saline matter of the two waters has nearly the same chemical composition. Great Salt Lake is low in carbonates and is associated principally with sedimentary

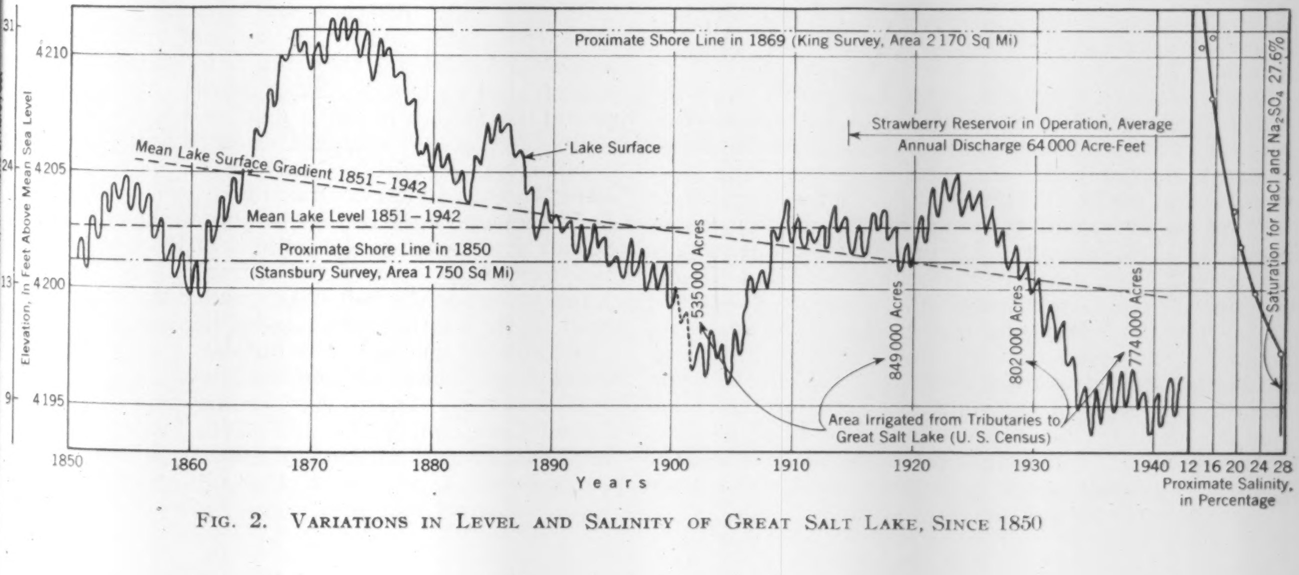
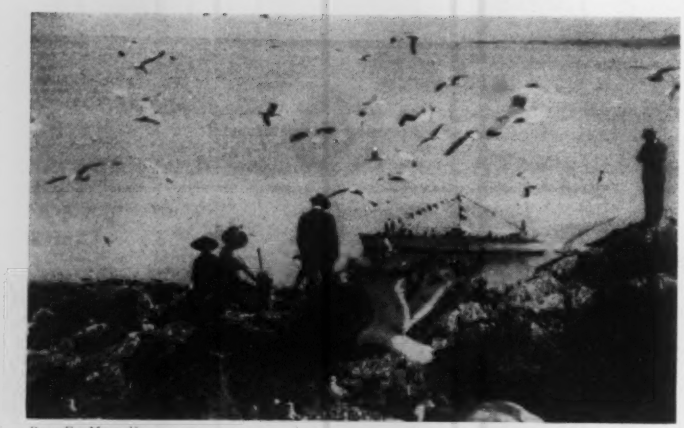


FIG. 2. VARIATIONS IN LEVEL AND SALINITY OF GREAT SALT LAKE, SINCE 1850



Ray E. Marsell

PLENTY OF FEATHERED COMPANY AT BIRD ISLAND

Herons, Pelicans, and Gulls Frequent This Spot

rocks. It is classified as a "saline" lake in contradistinction to an "alkaline" lake, which is high in carbonates and is representative of volcanic regions. Its water is a natural brine from which common salt and potash are being commercially extracted by the solar-evaporation process. It is estimated that the lake contains approximately 6 billion tons of salts, 85% of it common salt. The salt content changes nearly 1% for each foot of change in lake stage above the point of saturation.

About 11 miles west of Salt Lake City a large mill produces crude salt by solar evaporation. (See "Utah's Salt Industry," by G. M. Stratton, *Utah Engineer*, February 1929.) The brine is pumped from the lake into a flume, which empties into a series of ponds, where concentration and crystallization take place. Pumping begins in April and continues until the middle of September, except during storms. Evaporation is at the rate of about 5,000 gal per min for 24 hours a day from the middle of June to the middle of September. The pumping rate is keyed to the evaporation rate. The capacity of the plant is 50,000 tons of salt a season. Many grades are prepared for a variety of uses.

On the west side of the lake, the brines of the Salduro Marsh near Wendover are purified by solar-evaporation in successive concentration and crystallization ponds to a crystalline mixture of potassium and sodium chlorides, which are then separated by flotation. The final product is high-grade commercial muriate of potash, a valuable ingredient of mixed fertilizers. For 20 years, beginning in 1911, various agencies of the United States spent $2,000,000 searching for sources of potash within this country. (Minerals Yearbook 1940, U.S. Dept. of the Interior, Bureau of Mines.)

Fluctuations in the level of the lake result from the balance of the two factors, precipitation and evaporation—both highly variable. Practically a limitless number of balanced combinations are possible, considering that precipitation on the drainage basin ranges from 90 to 119% of the mean, and the evaporation probably ranges from 45 to 55 in. annually. During periods of abnormally low precipitation, the evaporation may be above normal and then the two factors combine to accelerate the rate of fall of the lake level; and conversely. It must be remembered, however, that the fluctuation in lake level will lag by several months, as regards the effect of precipitation, because of the time necessary for the runoff from the drainage basin to find its way into the lake. Irrigation, wind, and temperature also complicate the problem of evaluating the results of natural and human influences on the level of the lake.

Prior to 1869 when the transcontinental railroad was joined at Ogden from the east and the west, it was a slow and hazardous journey from Salt Lake City westward. Great Salt Lake and the stretches of barren salt flats west of it were forbidding obstacles to travel. The main travel route was around the north end of the lake. In 1903, by an epoch-making engineering feat, a great fill and trestle was built straight across the lake to shorten the main-line route by about 44 miles and eliminate troublesome grades around the north end of the lake. The relation of the two lines is indicated in Fig. 1.

The lake route, known as the Lucin cutoff, was constructed first as gravel fill across the shallow marginal parts of the lake and as trestle work through the deeper parts. Much of the trestle part was later filled in. Across the Bear River Bay, east from Promontory Point, some unexpected difficulty was encountered. Silt from Bear River had accumulated for centuries to form a bed of deep unstable mud. The railroad fill sank deep into this mud but did not readily reach a firm and permanent foundation. After the trains began running, successive "sinks" occurred. The weight of the fill, supplemented by the weight and vibration of the trains, seemed to break through some sustaining layer in the lake bottom and a whole section, track and all, would settle into the lake, while mud rose several feet above the surface and formed ridges on each side. Finally, after several years, the "sinks" were filled and became stabilized. The track across the lake is 4,217 ft above sea level, about 14.3 ft higher than the mean lake level of 1851–1942. It is level and approximately 35 miles long.

Because of the absence from Great Salt Lake of life such as is found in the ocean and bodies of fresh water, it has been termed "an aquatic desert," but myriads of water-fowl and shore birds are found on its islands, and along the northern, eastern, and southern shores. About the mouths of fresh-water streams and springs where the salinity of the lake is sufficiently reduced to permit the growth of vegetation, there are marshes where rushes, cat-tails and other plants make a favorable habitat for the heron family and many marsh birds.

For more than 100 years, to the knowledge of man, the Bear River marshes have been tenanted by a multitude of water-fowl. In recent years the area has been improved by the Federal Government and set aside as one of the nation's largest water-fowl refuges.

Lately some of the small islands in the lake have attracted attention because of their large colonies of nesting water birds—the American white pelican, double-crested cormorant, Treganza great blue heron, and California gull. All these are wide-ranging birds that have the status locally of summer residents. After wintering elsewhere, they begin to arrive in the Salt Lake Valley in late March or April. After the nesting period they depart in late August or early September.

Boating on the lake has never attained widespread popularity. As early as July 1871, "The Lady of the Lake," a little steam excursion boat, was brought to Salt Lake City and launched in the Jordan River; and after Saltair was opened in the summer of 1893, boating facilities were made one of the attractions of that resort. A few years ago the Salt Lake County Yacht Harbor was built on the southeastern edge of the lake near Garfield. It furnishes facilities for a number of privately owned small gasoline launches and sail boats.

It was early known by various names—Big Lake, Grand Lake, and by the Indians as Titchen-Pah, meaning "Salt Water." Bonneville was once considered as a name for it. Capt. John C. Fremont called it Great Salt Lake, a logical title finally made official.

he Shipshaw Hydroelectric Project, Canada

II. Design for Development of a Million Horsepower

By H. G. Acres, M. Am. Soc. C.E.

President, H. G. Acres and Company, Niagara Falls, Ontario, Canada

Designed under pressure to meet demands for war plant power, Shipshaw is a specific illustration of the results achieved by coöperation between owners, engineers, and contractors. Long after construction had begun, revisions due to unusual circumstances were made with no resulting postponement of the completion date. This article is condensed from a paper presented by Mr. Acres before the Engineering Institute of Canada, and is published with the coöperation of its publication, "The Engineering Journal." The historical development of the project was related by Mr. DuBose in the July issue.

THE power reach of the Saguenay River extends from tidewater, about a mile above he mouth of the Shipshaw River, o Lake St. John. The first stage of head concentration was at Isle Maligne, immediately below the utlet of this lake. At this point he Isle Maligne plant of the Sagunay Power Company was built. his plant operates under a normal egulated head of about 110 ft, has 20,000 kva of generating capacity nstalled, and commenced commerial operation in 1925.

The site chosen for the second stage of head concentration was immediately above the Chute à Caron rapids. Here a dam was built by the Aluminum Power Company, Ltd. see "Blasting a Pre-Cast Dam Into Place," C. P. Dunn, M. Am. Soc. C.E., Civil Engineering, December 1930), which ponded the water to the level of the Isle Maligne ailwater, about 22 miles upstream. This dam controlled normal regulated head of about 155 ft. Integral with he dam, a power house was built which now has 200,000 kva of installed generating capacity. This plant commenced commercial operation in 1930.

Location of the Chute à Caron dam and plant was such that there remained about 53 ft of free head between Chute à Caron tailwater and the head of tide. It was proposed ultimately to pick up this free head by a diversion canal starting at the Chute à Caron head pond and discharging through a power house and tailrace in he valley of the Shipshaw River, about half a mile bove its junction with the Saguenay.

The Shipshaw project, subject of several investigaions, again became active in 1940. A hydroelectric plant of roundly 1,000,000 installed horsepower of rated capacity, passing a prescribed ultimate volume of 50,000 cu ft per sec of regulated flow, was planned.

An unusual element of the Shipshaw scheme was the proposal to pass all the ultimate volume of regulated flow of the Saguenay River through the Shipshaw wheels, the idea being that whenever sufficient wheel capacity became available in the Shipshaw plant to pass all of the then-available regulated flow, the Chute à Caron plant would be completely bypassed.

The basic element of the design problem was to reconcile the compelling urgency for power to produce aluminum to be used by the Allied Nations with what would be considered a rational and sound design under more nearly normal circumstances. Secondary to this general limitation, there were two other abnormal and uncontrollable factors. One was the growing scarcity of metal products for other than direct war purposes, and more particularly steel plate. The other was the fact that the Shipshaw plant, as geographically located, would be subject to the hazard of enemy bombing attack.

FIVE DAMS REQUIRED

During the spring and early summer of 1941, soil surveys and analyses of the overburden material were made on and adjacent to the site of the development, the general layout of which is shown in Fig. 1. Material was found that was suitable for the construction of the impermeable core of a dry-rolled fill. The available yardage, however, fell far short of what would have been required for the three dams in question, and by coincidence was found to be sufficient in quantity only to cover the estimated yardage required for Dam No. 5.

For this reason Dams Nos. 2 and 3 were planned as concrete structures bonded throughout to the underlying ledge. Dam No. 5 was designed as an embankment with an impermeable core of dry-rolled fill, and with a core wall bonded to rock and extending slightly above the natural elevation of the saturated overburden at the site.

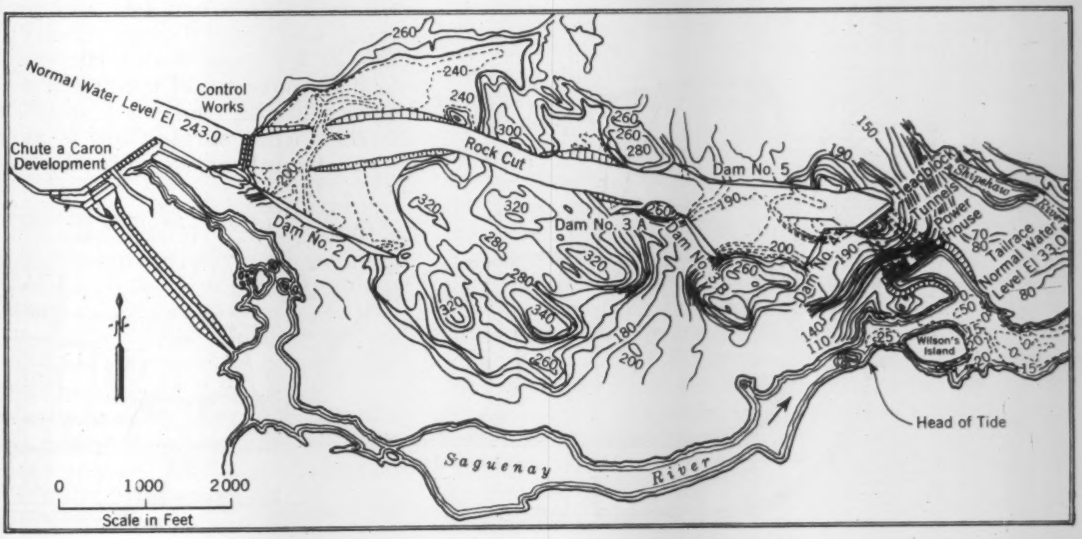
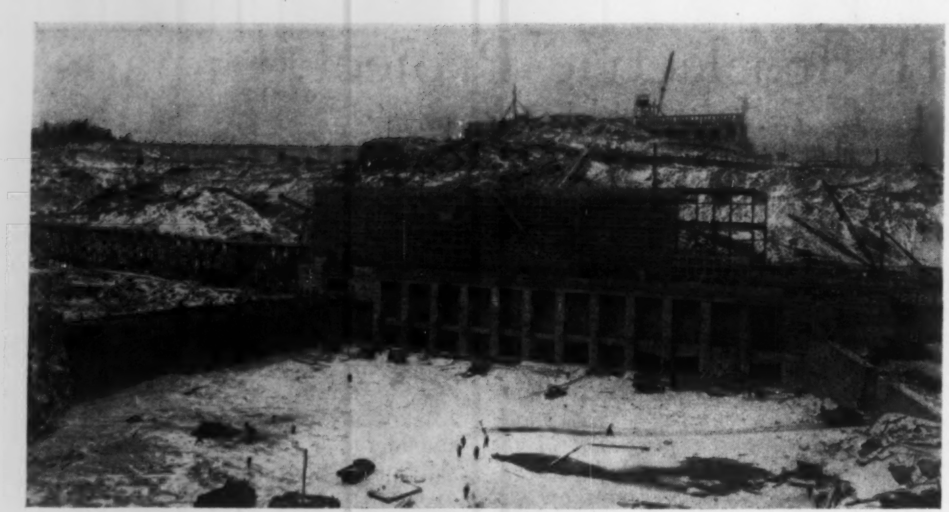


Fig. 1. General Layout of Shipshaw Project



SHIPSHAW POWER HOUSE, LOCATED ON A ROCK LEDGE AT FOOT OF A RIDGE

This design was prepared on the assumption that there would be two summers (1941 and 1942) sufficiently warm and dry to allow the structure to be completed before the fall freeze-up of 1942. The stripping of the overburden and excavation for the core wall were actually commenced on the basis of this design.

As the season of 1941 advanced, however, three adverse conditions manifested themselves in connection with the working schedule for this embankment. In the first place, an acute shortage of earth excavation and handling equipment developed. In the second place, largely because of the lack of sufficient equipment, it transpired that the dry-rolled fill material would have to be removed before it could be placed in Dam No. 5, thus making it necessary to store this material in a specially prepared dump and thereafter to rehandle it if it were to be used at all. Third, it was found that the most suitable location for one of the largest central mixing plants ever built must be immediately adjacent to the site of Dam No. 5, and that, with its auxiliaries and approaches, it would actually encroach upon the site of the embankment.

The final result was that Dam No. 5 was built in the form of a concrete bulkhead. None of the excavation work that had previously been done in connection with the embankment was lost, however, as the concrete structure was built on the same site, and the removal of the overburden was necessary in any event to expose the ledge for the base of the dam.

FOREBAY ENTRANCE CONTROLLED

As will be seen from Fig. 1, the entrance section to the canal and the section through the rock ridge are the only two regular prism sections in the whole waterway, and the design provided for a control structure in the upper prism section. This control structure was included in the design so that the Shipshaw canal could be unwatered without pulling down the Chute à Caron head pond level by about 35 ft. This would have had the result of wasting about four billion cubic feet of expensively stored water and destroying the production of the Chute à Caron power plant.

At least three happenings might make it necessary to unwater the canal: first, a dangerous leak toward the river through the underlying ledge; second, the necessity of future repairs to the concrete work in the zone of the water line; and third, a bomb hit on the headblock.

The control structure was adapted to serve two other purposes, as follows:

1. A long slide across the canal area which it was necessary to maintain. The only practical means of so doing was to divert the slide and carry it across the narrow section at the canal entrance.
2. A provincial highway crossing the canal, which had to be maintained. Similarly, the only practical means of so doing was to carry it across the narrow section at the canal entrance on a bridge.

ICE PROBLEM

No surplus water is available for the sluicing of floe ice or frazil. There is a continuous ice cover throughout the winter on the 22-mile reach of headpond between Chute à Caron and Isle Maligne, and consequently no frazil, and not sufficient velocity to move any large volume of ice cover during the spring break-up. Such ice as does move out of the headpond is disposed of through sluices in the Chute à Caron Dam during the spring freshet, when surplus flow is available for the purpose. This means that, so far as the ice problem is concerned,

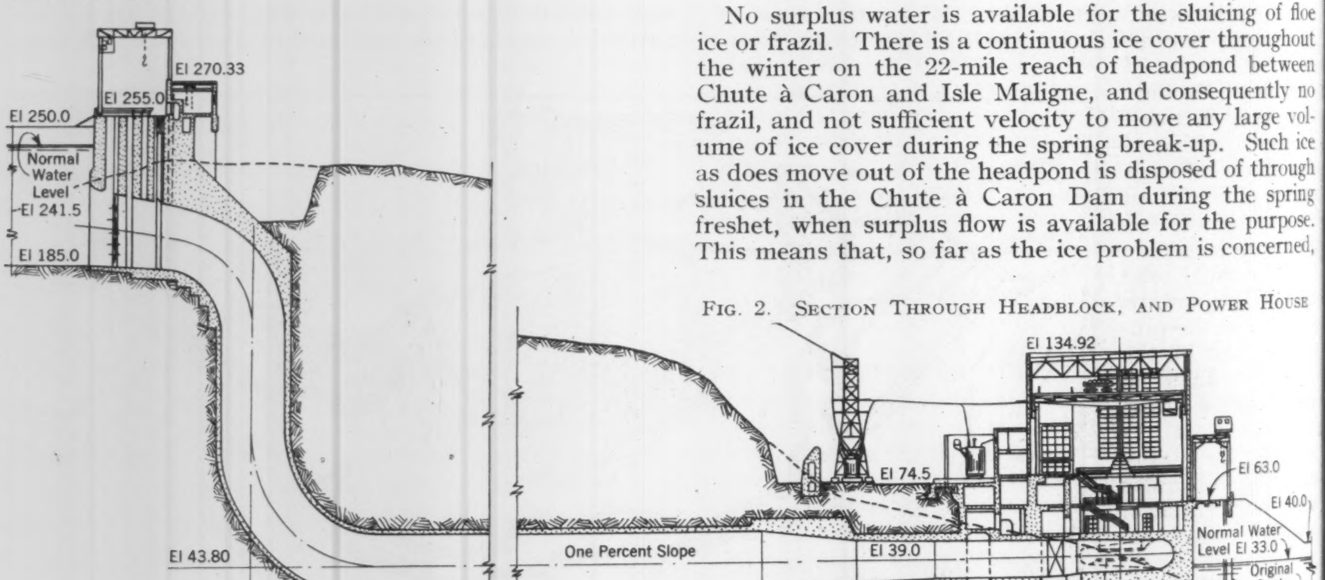
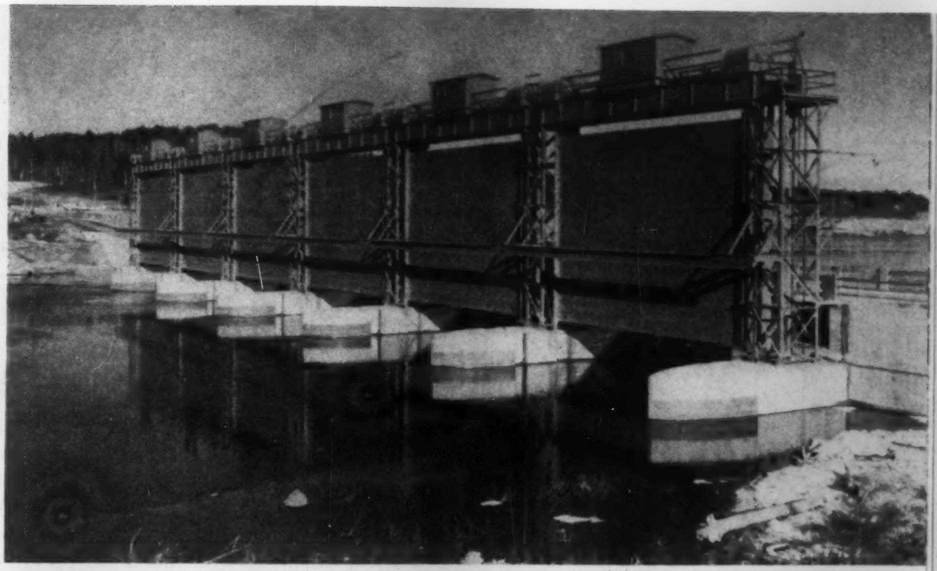


FIG. 2. SECTION THROUGH HEADBLOCK, AND POWER HOUSE

only a 7,350-ft reach of waterway between the Shipshaw control works and the headblock is involved. A large part of the area of this reach will also have a permanent ice cover, with open-water areas.

Under the circumstances, no provision was made for sluicing ice out of the Shipshaw forebay, and three openings only were left—one in the crest of Dam No. 2, one in the crest of Dam No. 5, and the other in the crest of the north wing-wall, for the purpose only of disposing of occasional accumulations of pulpwood and other floating debris.


Control Works at Entrance to Shipshaw Canal for Unwatering of Canal

The second problem of design was to fix upon the sites for the headblock, water connections, power house, and tailrace. An exposed ledge of rock at the foot of the steep river slope provided ample room for a power-house substructure and a considerable part of the tailrace. Selection of this site meant that the whole area of the power-house location, and about 98% of the tailrace earth and rock could be taken out in the dry, independent of season. Only the easiest type of earth excavation was required, and only a comparatively small amount of rock excavation, to prepare the power-house foundations for the pouring of the substructure concrete.

HEADBLOCK AND WATER PASSAGES

The rim of the escarpment immediately above the tailrace location juts out over the valley in the form of a hogback ridge with a steep drop towards the river, its south flank sloping into a ravine and its north flank dropping down towards the Shipshaw River. This meant that the headblock would have to be placed transversely across the crest of the ridge at some point where there was sufficient lateral space to accommodate the intakes of whatever type of water passages might be adopted. The location of the headblock therefore was a controlling factor in determining the length of these water passages.

Obviously, there were only two types of water connection to be considered—exposed steel penstocks and a combination of shaft and tunnel. It was at once evident that if these water passages took the form of exposed steel penstocks, the whole crest and face of the escarpment would be shattered and broken up to such an extent that it would probably be necessary to protect the whole of the exposed surface with concrete, and also to provide expensive frost protection. Furthermore, as regards use of critical war materials, these penstocks would require about 8,000 tons of steel plate, and priority deliveries would be unpredictable. As to war risk, they would be particularly vulnerable to bombing attack.

The alternative was to install the water connections wholly in rock. All known conditions were favorable to this arrangement. The hard, brittle rock was suitable for the pattern drilling and comparatively light shooting which is customary on this class of work; neat lines would be preserved with a minimum of overbreak and with an absence of "squeeze." These advantages were likely to be enhanced because there was to be sufficient working room to use circular headings.

Shown in Fig. 2 is a typical cross-sectional view of the headblock, tunnels, and power house as built, showing the adopted alternative of combined circular shafts and tunnels for water passages. There were six of these, concrete lined, each 30 ft in finished inside diameter.

A plan layout of the shafts, tunnels, and connections to the main units appears in Fig. 3. As will be seen, all the shafts are vertical and have the same lateral spacing, but the tunnels fan out from the lower shaft elbows to accommodate the main-unit spacing in the power house. Steel plate liners 30 ft in diameter, in lengths varying with the natural contours of the overlying ledge, were inserted in each portal and sealed in with concrete. These liners were inserted to a distance which would give the inner end not less than a 40-ft coverage of undisturbed ledge.

The outer extremities of these liners terminated in a wye, with two 18-ft diameter branches, from which short lengths of penstock of the same diameter extended to the point of connection with the turbine scrollcases. All this steel-plate work was welded and subsequently embedded in a reinforced-concrete envelope, with footings on solid rock. The average length of water passage from headblock to scrollcase is approximately 740 ft.

GENERATING EQUIPMENT

All water that would ultimately be available for power production was to be passed through six tunnels, each of which had two penstock connections at its outlet. These twelve main units would constitute the ultimate limit of installed capacity, and each turbine would have a rating of 85,000 hp in round figures.

The matter of the number of units, and of individual unit capacities had to be decided with regard to the fact that the bypassed Chute à Caron plant could be considered as a spare unit having a capacity of 200,000 kva. This assumption, however, was subject to the reservation that this plant could not function effectively as a spare on short-period outages unless some considerable part of its installed capacity were kept warmed up and turning on speed-no-load. This would involve not only the permanent maintenance of an operating staff at Chute à Caron, but also the use of stored water under a 50-ft loss of head and at an extremely low efficiency. On the other hand, Chute à Caron could function as a spare with full effectiveness on arranged outages at Shipshaw, such as periodic shutdowns for refitting and overhauling, and for minor running repairs.

These two considerations relative to the use of Chute à Caron as a spare had the following reactions. In the first place it made it advisable, independent of Chute à Caron, to have sufficient capacity in the individual units at Shipshaw to bridge a short period of single-unit outage by temporarily overloading the other units. In the second place the availability of 200,000 kva of spare capacity at Chute à Caron made it permissible to consider arranged outages in terms of pairs, instead of single units.

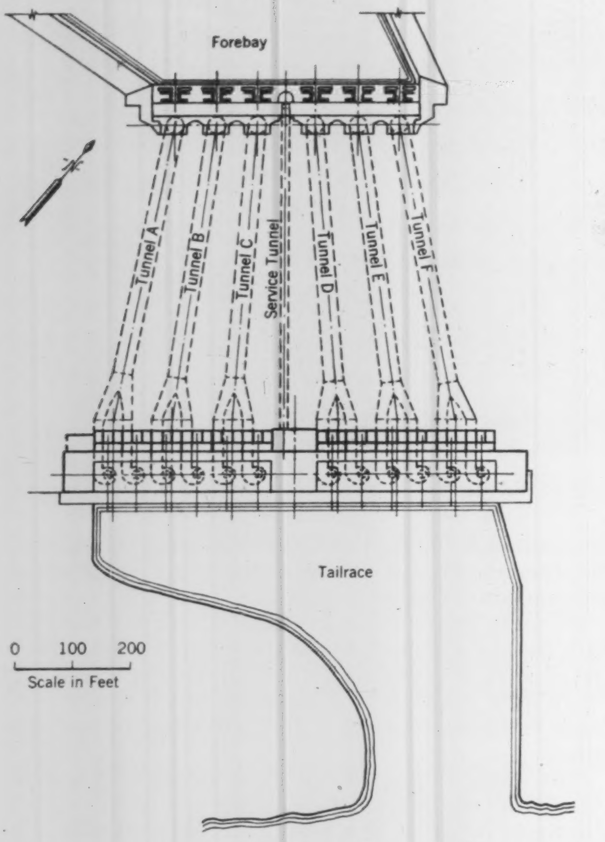


FIG. 3. PLAN OF RELATIONSHIP BETWEEN HEADBLOCK AND POWER HOUSE

This latter consideration was important, because it made it possible to take a unit out by dropping the headgates and unwatering the whole connecting system. Furthermore, it obviated the necessity of installing penstock valves, thereby eliminating a certain amount of hydraulic loss, a troublesome item of maintenance, and a very material element of capital cost.

Turbine specifications called for vertical-shaft, single-runner, Francis-type units of 85,000-hp capacity at or near best gate, operating at a 60-cycle speed of 128.6 rpm. For the first time, completely welded turbine scrollcases were installed in the field, including the connection of the plates to the cast-steel flanges of the speed-ring. The inlet diameter of the scrollcase is 16 ft, and its maximum overall diameter is 49 ft 8 in.

Specifications called for 60-cycle, 3-phase, 13.2-kv generators having a capacity of 75,000 kva at 80% power factor and 60 C maximum temperature rise. Thrust bearings were placed on the upper bearing bracket, and above were direct-connected main and pilot exciters.

CONDITIONED AIR COOLING SYSTEM

One special requirement of the specifications was that these generators should be ventilated by a closed system of water-cooled air, for two reasons. Owing to the large capacity of these generators, it was desirable that they should have the advantage of an artificially controlled ambient, on account of the wide range of natural temperature variation in the Lake St. John district, and the more so because of the comparatively low summer temperature of the water supply drawn from the deep 22-mile reach of the Chute à Caron headpond.

If, for instance, the summer ambient can be kept down to a maximum of 70 F, by water-cooled recirculated air, as compared with a natural ambient of, say, 80 F, an additional 377 amp per phase can be carried within the limit of the guaranteed maximum temperature rise of 60 C. This would add 8,615 kva to the output of each generator—a total of 99,150 kva for the 12 units.

A second reason is that a closed system of ventilation is free of dust, grit, insects, and other matter light enough to be moved. In an open system this matter has a tendency to lodge in the smaller ducts and on heat-softened insulation, so that in the course of time the heat-dispersal efficiency of the generator becomes impaired to a point where it suffers an appreciable reduction in output, for any permissible limit of temperature rise.

The use of water-cooled generators was important from the purely structural standpoint. Otherwise it would have been necessary to lengthen the turbine shafts about 8 ft, and to install an additional reinforced-concrete floor over the whole area occupied by the main units, in order to accommodate the duct system which would have had to be provided.

The Canadian Westinghouse Company and the Canadian General Electric Company divided the order for the ten new generators with the understanding that all differences in electrical design and characteristics would be satisfactorily compromised, and that all generators, when installed, would be visually identical in outline and exterior dimensions. The generators have housings 36 ft square, and the height from the floor level to the top of the pilot exciter is 23 ft 1 in. The water cooling units are located in each of the four corners of the square housings.

AUXILIARY EQUIPMENT AND SERVICES

The basic consideration underlying the layout and the design of the auxiliary services was the length of the power house, which as designed was 837 ft. On this account it was obvious that all auxiliary services involving the pressure or gravity flow of oil, water, air, and electricity should have their origin or outlet, as the case might be, as near as possible to the transverse center line of the power-house building. The primary element of the design was therefore the location of a control section, centering on the transverse power-house center line. The control board, control wiring, relays, sump pumps, compressors, storage batteries, transformer, and switch-oil transfer and filtering equipment, service-water strainers, power-house service board, ventilating equipment, elevators, and offices of operating staff were all installed in this control section, with the main units spaced right and left in groups of six each. Similarly, all gravity water drainage was outletted into a main sump installed in the basement of the control section.

One feature of the auxiliary service layout is the elimination of all brass and other pipe for the oil-pressure supply and gravity drainage. In place of the usual oil-piping system, four steel tanks are provided, each of 20-bbl capacity, one pair being used exclusively for clean oil and the other pair exclusively for dirty oil. These tanks are provided with bails so that the auxiliary hook of the power-house crane can pick them up and drop them through a hatch onto the turbine deck beside any of the 12 main units. When so deposited, the clean-oil tank would be full and the dirty-oil tank empty. Thereafter, in the case of a generator thrust bearing, the procedure for changing oil would be to attach a flexible hose to the drain outlet in the bottom, and drain the oil into the dirty-oil tank. The next step would be to make use of a small pump attached to the clean-oil tank to get the clean refill into the bearing. In the case of a turbine guide bearing, the procedure would of course be reversed. The dirty oil would have to be pumped out and the clean oil run in by gravity.

Municipal Postwar Planning

The Attorney's Point of View, in Review and Preview

By EDGAR A. B. SPENCER *and* ROBERT E. DEMPSEY

ATTORNEYS, RESPECTIVELY, NEW YORK, N.Y., AND THE TOWN OF CORTLANDT, WESCHESTER COUNTY, N.Y.

WHILE there is wide unanimity of opinion as to the need for postwar planning, there seems to be considerable disagreement as to the meaning of the expression, the purposes of such planning, the ends to be accomplished, the conditions that will have to be met when hostilities cease, and the proper means for accomplishing desired results. Experience in the PWA era, and studies since, are here presented to help in clarifying the term.

POSTWAR planning has been so much under discussion that the *New York Times* has devoted about three-quarters of a column a day to the subject. Other newspapers are also giving it a great deal of publicity. Some people use the term "postwar planning" as synonymous with the planning of a relief program for the days directly following the war. Others use the expression in the accepted engineering sense, but stress the fact that special consideration must be given to the immediate postwar era. The second is by far the sounder approach, as can be readily demonstrated.

POSTWAR UNEMPLOYMENT AND PUBLIC WORKS

Whatever, the viewpoint, there seems to be a general realization that upon the cessation of hostilities in whole or in part, there will be a grave unemployment problem (or "lag" as it is sometimes called) until private industry can be converted to peacetime production. How long this lag will be, and how severe the unemployment, cannot be predicted with any reasonable degree of certainty.

It is generally agreed that the carrying out of public works by the construction industry during this period will be the quickest way to provide temporary employment. This point of view is entirely sound, provided that the public works have been previously planned, since they require no tooling-up period, are generally of comparatively short duration, and employ large numbers of unskilled and semi-skilled labor. They also are ideal since about 60% of the cost is for direct labor and, if material charges are properly analyzed, it is found that about 90% of these also goes ultimately to labor.

Providing employment during this lag is one, if not the principal, aim of most postwar planning. Since hostilities may end with little or no advance warning, postwar plans should be made now.

Probable unemployment is largely a local problem which public officers must consider at once. Unfortunately they have no federal or state laws to guide them in planning emergency public work and they must therefore rely on their own judgment as to the conditions to be encountered, the legislation which by that time will be enacted or in process of enactment, and the probable means that will be available to provide the employment.

The Federal Government has one plan of emergency construction of public work for this period—highway construction with state cooperation, expected to cost about $250,000,000. But it must be remembered that the Federal Government operates in a restricted field and could not possibly develop and carry on enough public work to meet the unemployment problem to any appreciable extent. If this problem is to be met in any substantial degree by emergency public works, it is the states and their political subdivisions which must carry them out.

© *Fairchild Aerial Surveys Inc.*

AIR VIEW OF TALLMANS ISLAND (NEW YORK CITY) SEWAGE TREATMENT WORKS

This PWA Project Is One of the Best Developed of Its Kind

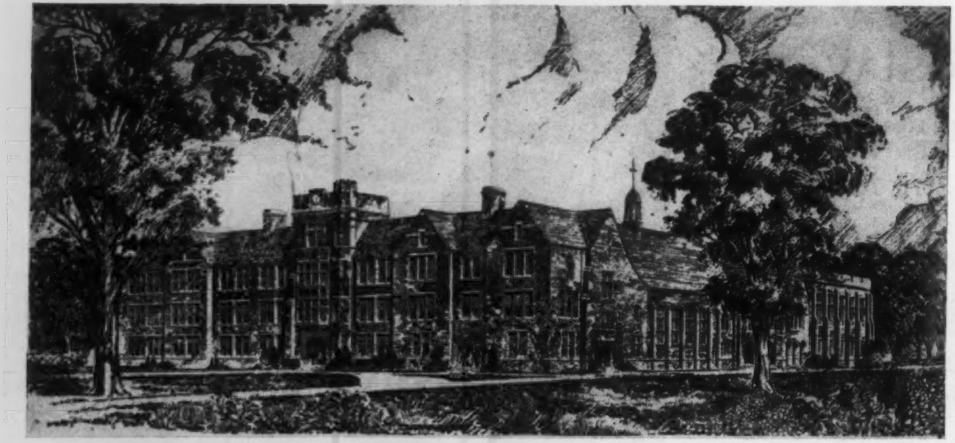
In most localities a public works program can be developed which not only will help provide employment but also will make provision for needed repairs to public structures (neglected because of the war) and for the extension of public facilities to meet the changed conditions of recent years. Many items on such a program can be made self-supporting and self-liquidating.

It is of course the dream of every planner that even though the motive for carrying on an emergency public works program is to provide employment, his program will be so well developed and carried out that the result will be, to the full amount expended, a corresponding increase in the national resources by construction of public utilities of real value. This dream can only be realized to any appreciable extent by adequate study and planning. The sooner it is carried out, the better the result will be.

Lest anyone doubt the magnitude of the problem to be faced, it might be noted that *Reports for Business Executives* (issue of February 3, 1944) states that "about 8,000,000 more workers may be looking for work than there are jobs to be filled." If this prediction turns out to be accurate, more than a short-term program will be required.

EMPLOYMENT IN THE CONSTRUCTION INDUSTRY

In assigning to the construction industry the role of relieving unemployment, it must be borne in mind that even now the industry faces a steadily mounting unemployment problem. Many workers who have not been able to obtain employment in the industry have drifted into war plants where they can use their previous training



DOBBS FERRY, N.Y., HIGH SCHOOL—ARCHITECT'S DRAWING
First School Built Under PWA Auspices in State and One of Its Finest School Structures

to some extent. The cessation of hostilities will turn out many of these workers to join their unemployed brethren.

We are apt to think of soldiers and sailors as fighting men. So they are, but many are not fighting with rifles—they are also on landing fields and beaches, using bulldozers and other construction equipment now of such great usefulness to our armed forces. These men were not all recruited from the construction industry; many were drawn from the ranks of unskilled labor and frequently from the white-collar group and were trained by the Army and Navy. But they are now construction men, and compare high union wages with the lower rates usually paid for unskilled and clerical employees. Naturally, many will seek reemployment in the construction industry.

The present state of the construction industry can best be shown by comparing the national construction figures for the years 1942 and 1943, and the New York City building figures from 1930 to 1942. In 1942 there was $13,400,000,000 of new national construction, mostly of war facilities, and in 1943 about $7,400,000,000, most of it in the first half of the year. In New York City, building construction dropped from $946,916,566 in 1930 to $191,130,176 in 1940, and to $54,450,196 in 1942.

While the 1940–1942 drop in building in New York City was caused by war conditions, which will change when hostilities cease, causing a revival of building, many who were in the building industry will be unemployed.

It is the general feeling of public officers that when taxes are high (as they certainly are now) they should keep their capital expenditures to a minimum, unless they can point to a real justification for making capital expenditures. It was found in the years 1933 to 1938 that the receipt of federal grants (that is, the payment of part of the cost of a project by the Federal Government) was sufficient inducement for voters to approve, and for public officers to undertake, public works of vast magnitude. During that period PWA grants for non-federal purposes of $1,512,034, 557 induced the construction of $6,128,952,061 non-federal projects.

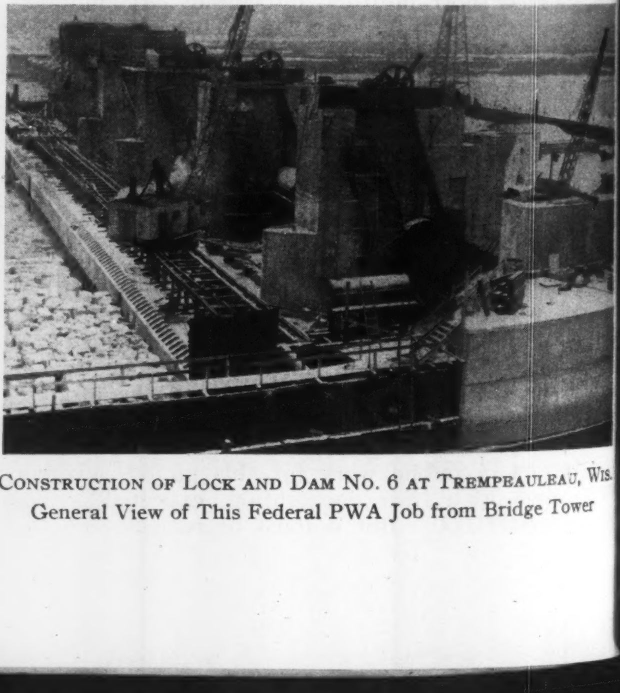
With New York City's present estimated population of 7,625,000, of whom about 200,000 normally have looked to the construction trades for employment, the city's program provides for the annual employment of 250,000. While severely criticized, this figure seems to be a conservative one. Although many construction workers will find private employment in the industry, the city's program is designed to provide employment for the remainder, as well as for many who have not been construction workers, and for returned soldiers who have been trained in construction skill by the Army and Navy. A program calling for the direct employment of about 3% of the population in an emergency public works program appears sound and even conservative. This program could be used to the extent necessary.

At this writing (May 1944) there is no provision for federal participation in the cost of local postwar emergency construction (except in connection with the federal highway program), nor is there in New York any provision for state participation. Probably municipalities will not have to bear the total cost of emergency construction, for there will be such a demand for a federal program that Congress will no doubt authorize federal grants for such construction.

Some of the states are now building up large surpluses for postwar use. For example, New York State has a current surplus of about 140 million dollars which has been earmarked for postwar purposes; New Jersey similarly has one of $25^1/_2$ million; and California is expected to have 200 million, which probably will be similarly earmarked. How these monies will be spent has not been determined but the probabilities are that they will be used for state public works and other state purposes, and that little or none will trickle back to the political subdivisions of the states.

BASIS OF THE DEMAND FOR FEDERAL AID

It has been suggested that the Federal Government should bear a substantial part of the cost of postwar emergency construction. The equities of this suggestion are that (1) since the Federal Government took the workers out of peacetime gainful pursuits, it should aid in returning them to gainful occupations; (2) since by its hold on the banking system, the Federal Government controls, to a large measure, the flow of money and the rates of interest paid on public loans, it should use this control and low money rates for the benefit of communities that are trying to help solve an obvious unemployment problem; and (3) only the Federal Government can dovetail thousands of separate community plans into one over-all, well-coordinated plan.



CONSTRUCTION OF LOCK AND DAM NO. 6 AT TREMPEAULEAU, WIS.
General View of This Federal PWA Job from Bridge Tower

How much federal participation will be authorized no one can say. Mayor La Guardia of New York City has said that the Federal Government should have a program of 15 billion dollars, which would be slightly more than the former PWA and WPA programs combined. Other less expensive programs have been proposed. Judging by the past, it is reasonable to expect that Congress will do nothing concrete about postwar planning until the end of hostilities is definitely in sight.

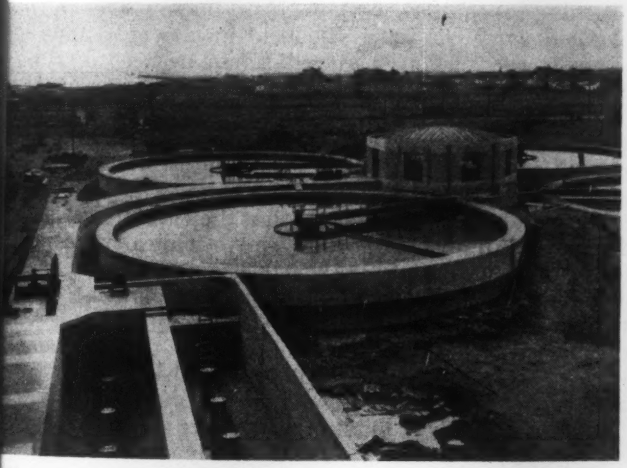
Based on studies of work done for about twelve New York State municipalities from 1933 to 1938, it is our opinion that construction under the PWA plan cost from 12% to 20% more than it would have if done without federal participation. Of the cost of the early work, the Federal Government paid 30%. Of that done in the later years, the Federal Government paid 45%. The general average of extra cost ran about 12%, the 20% case being caused by PWA insistence that the original bids for one contract be rejected.

The excess cost we attribute to (*a*) extra clerical work required, (*b*) excessive inspection (the curse of a great deal of government work), and (*c*) lack of flexibility in the conduct of the work. In any event, all the projects being necessary improvements, the municipalities concerned gained at least 18% of the cost of the improvement, and the public officers and taxpayers seemed satisfied with the results accomplished. A new emergency public works program should produce results at least as good.

Foundation for Criminal Courts Building, New York City
In Its Entirety, One of the Largest PWA Projects in the State

EXAMPLES OF POSTWAR PLANNING

While this article is not intended as a survey of postwar planning, the following illustrations show, for New York State, New York City, and Massachusetts, what has and what has not been done.



Coney Island (New York City) Sewage Treatment Works
Flocculator Channels, Settling Tanks and Clarifiers, and Sludge Pump House

By Chapter 660 Laws of 1942, the Legislature of the State of New York established a Postwar Public Works Planning Commission to coordinate, stimulate, and aid in preparing a program of worth-while public works which could be put into immediate and effective operation when hostilities cease. It has the power to aid municipalities in planning by way of grants up to one-half of the cost of the necessary plans, to be made ready as nearly as may be for immediate letting. The Commission has made such grants in the case of 1,149 projects.

As of January 1, 1944, it has received for study 3,407 municipal plans having an aggregate estimated construction cost of 328 millions from 378 political subdivisions of the state, excluding the City of New York. In addition the state itself has prepared 110 authorized projects having an aggregate construction cost of 21 millions. Also the state has under study other long-range plans, of which the following are a few:

Highway Program. 5 years, estimated 780 million. (For two years of the program, funds are now available.)

Low-Rent Housing. 300 million has been authorized by the Constitution and one-half released for contract purposes by the Legislature. Of this, 90 million will be contracted for immediately after the war.

This is a splendid program as far as it goes, but there is another view of the situation which is not reassuring to some localities. There are 57 counties, 934 towns, 575 villages, 62 cities, and hundreds of school districts and special improvement districts in the state. Of these only 378 have made application to participate in the state's planning program. Chapter 696, Laws of 1943, authorized municipalities to issue capital notes for 50% of the cost of preparation of plans for postwar projects. So there is no legal impediment to such planning.

In the case of New York City, the Mayor has announced that his city is ready with 800 millions worth of public works. (See "Manhattan Prepares for Postwar Construction," by Dean G. Edwards, M. Am. Soc. C.E., and "Postwar Sanitary Engineering Work for New York City," by Richard H. Gould, M. Am. Soc. C.E., in Civil Engineering for August 1943 and May 1944, respectively.) About 50 million a year is available for capital expenditure, not counting federal aid. With federal or state grants on a 50% basis, New York City will thus be able to carry on about 100 million dollars worth of construction work for about eight years. On this basis its plans have been made.

VAN CORTLANDTVILLE, N.Y., SCHOOL—TYPE OF THE SMALLER PWA STRUCTURE

Hundreds of Comparable Schools Were Built from 1934 to 1938

The New York City program has been developed to such an extent that contract plans, specifications, and tentative contract documents have been drawn for about 30% of the proposed projects; the engineering has been done and tentative contract documents are being drawn for at least 20% more; engineering for about 25% of the projects is under way; and only about 25% of the projects still remain in the preliminary stage. The City could proceed with its program almost immediately.

Massachusetts has now planned for projects expected to cost 250 millions, made up as follows: state projects, 50 millions; and city and town projects, 200 millions.

A TASK FOR ENGINEERS AND PUBLIC OFFICERS

It seems probable that if federal aid is not to be extended, the emergency public works program will be reduced one-half. In fact, a failure of federal participation might serve to actually discourage public works far beyond the amount of the hoped-for funds. However, New York City is drawing up its contract documents so that it can proceed with or without federal aid.

Development of the physical aspects of any construction plan is of course the work of architects and engineers. Similarly the authority for preparing plans, for fitting proposed plans into the public scheme of things, and for obtaining fiscal support for accepted plans is of course the duty of public officers. But by far the most important work, also to be performed by public officers, is the task of convincing the electorate that their planning is sound and in the best interest of the public even if considered from a narrow local viewpoint.

There is also much detailed work that should be done by public officers now. Municipal records should be brought up to date and made available for immediate use. These include records of corporate boundaries, copies of charters or basic creating acts with amendments to date, assessment records for about five years, tax-collection records for about five years, and records of the funded and floating debt for five years.

Where voters' approval of emergency construction programs is required, propositions should be prepared and submitted to the electorate as soon as possible. It is generally feasible to so word submitted propositions that, if carried, they can be used no matter whether federal aid is obtained or not.

Contract plans, specifications, and documents should be drawn now, so that they will be ready for immediate use. Since it is not known whether federal aid by way of grant will be available, such contract documents should be so worded that they will satisfy present federal requirements and can be amended quickly to meet the requirements of any future law.

If the PWA program is a criterion of what is apt to happen, the attorneys for states and political subdivisions must be prepared on very short notice to (*a*) study and understand the essential basis and workings of any federal legislation for an emergency public works program; (*b*) improvise applications for aid; (*c*) check his own clients' fiscal structure to be sure that it can enter into the proposed project, that it can finance its share of the cost of the project to be borne by it out of revenues or bonds; (*d*) quickly adapt existing contract documents for planned public work to the requirements of the new legislation; (*e*) if special local assessment rolls are required, outline their form and guide public officers in preparing and adopting them; and (*f*) if special legislation is required on account of charter or debt requirements, see that it is drawn as soon as possible.

AIDS TO PRIVATE INDUSTRY

The following observations, although not strictly applicable to postwar planning, are certainly pertinent to the problem of unemployment.

1. Employment on emergency public works is a poor substitute for employment by private industry, and has been able to furnish only a fraction of the employment that flourishing private industry has. For example, buildings costing 221 million dollars were built in New York City in 1920, while buildings costing 947 millions were built in 1930. Not all persons in the public eye are convinced that an emergency public works program will completely care for all postwar unemployment problems. Some people are inclined to cast doubt on the entire program. Their doubts as to the quantity of public work to be undertaken merit study, but their doubts as to the validity of the use of public works to help solve unemployment during the immediate postwar period hardly seem sound. Others believe that a federally sponsored plan is unnecessary, on the theory that once the war is over we shall have a building boom of such magnitude that "construction" in the first ten years after the war will be 5% greater than in the boom years 1920–1929. This is not a valid objection to present planning of public works. If a boom develops, the planned work can easily be held in abeyance until the boom is over, when there will again be need of such a program.

2. Public officers should encourage private construction in their localities by (*a*) encouraging private enterprise; (*b*) encouraging home building; (*c*) avoiding unfair and excessive taxes and assessments; (*d*) examining their zoning ordinances to see that they are equitable and do not unduly restrict normal, worth-while development; (*e*) examining their building codes to see that they are modern and do not impose unnecessary burdens on construction; (*f*) interesting labor leaders in their plans so that construction can be so carried out as to insure proper working conditions for employees with a minimum of onerous conditions and unnecessary work stoppages.

Whether postwar planning develops into a "leaf-raking" proposition or a skillfully devised measure for national development will depend on the soundness of the action of local and state public officers and their willingness to cooperate to create a real plan for the physical improvement of the country. In any event, it is well for all to realize that an unemployment problem will exist after the war and that unemployed people must be cared for. The plan that calls for the greatest ultimate return for the dollars spent is the one to be adopted.

Survey of Navigation on the Ohio River

Forty-Six Locks and Dams Maintain Navigable Depth from Pittsburgh to the Mississippi

By **Frederick B. Duis**, M. Am. Soc. C.E.

"So long as water runs down to the sea, so long as man is possessed of industry and craftsmanship, so long as winter snows and summer skies nurture the growing crops—so long will our Ohio River continue as the great arterial highway to the outside world." This thought, expressed by the late R. R. Jones while he was Principal Engineer in the Cincinnati U. S. Engineer Office, indicates the ever-present need for river transport facilities to serve the Ohio Valley. The development of the river by construction of dams and locks and channel improvements, with resulting increase in commerce, is related by Mr. Duis. This paper has been condensed from his address before the Cincinnati Section.

NAVIGATION probably existed on the Ohio River centuries before the beginning of recorded history. The Indians went up and down it in canoes long before the white explorers came, and we may assume that long before that the race or races which existed in the valley and of whom we find evidences but no historical data, did the same. However, it was not until shortly after the close of the Revolutionary War that any appreciable amounts of goods or passengers were carried on the river. It then became an artery for travel from the Eastern settlements that had merged into a union of states, into the great unsettled wilderness. North of the river the forests were broken only by the narrow trails of wild animals and Indians, while south of the river, such pack-horse trails as led to the pioneer settlements were difficult and dangerous to travel. Barring shipwreck on the way, the Ohio River furnished a route easier and less dangerous than those by land. For the vast migration westward, this was the "River of Destiny" for millions of home seekers. The descent of the river by the pioneer settlers in the valley constituted a most remarkable journey. It was made in crude small boats and even on rafts, hastily constructed for the occasion, carrying not only the pioneers but also their household goods, farm implements, horses, cattle and other animals, besides provisions for the travelers and for the animals.

EARLY RIVER TRAFFIC AND "STEAMBOATING"

During the early pioneer period, the small boats using the river were without power except for an occasional sail. Most of the early commerce, therefore, was downstream. The early settlers had no adequate near markets for their surplus produce and manufactured goods, and so shipped such commodities down the Ohio and the Mississippi rivers to the communities along the lower Mississippi, especially to New Orleans, where much of it was reshipped to foreign countries. At the end of their journey to market, the Ohio Valley shippers usually disposed of their boats, and frequently walked back home.

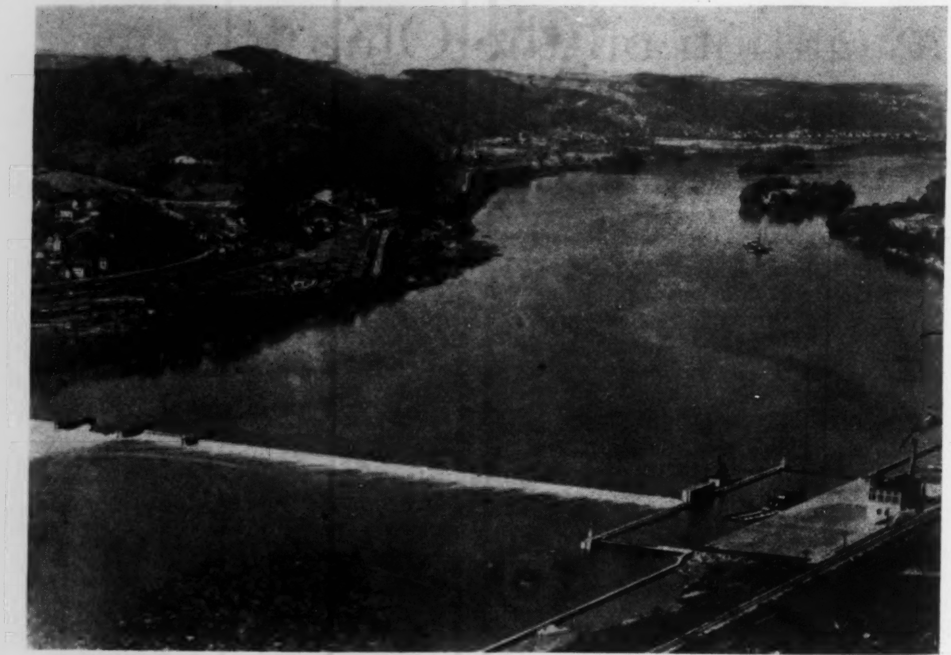
A mighty impetus to navigation on the Ohio River occurred with the advent of the steamboat. The first one on the river was the packet boat *New Orleans*, built in Pittsburgh in 1811. It was a 300-ton sidewheeler, 138 ft long. Evidently the owners did not have too much confidence in this new type of power for the boat was also rigged with masts and sails. There were 63 steamboats in 1819, 232 in 1832, 450 in 1842, and 750 in 1855 plying the Mississippi and Ohio rivers and connecting waterways. Of the 348 steamboats built on these rivers in the twenty years from 1811 to 1831, 111, or almost one-third, were constructed in Cincinnati.

By the middle of the nineteenth century, a considerable mileage of railroads had been constructed in the valley and offered serious competition to river commerce. Low stages in the river frequently caused suspension of navigation for long periods, making it difficult and finally practically impossible for the river traffic to compete with the greater dependability of the railroads. In addition, there were large losses in boats and cargoes resulting from obstructions in the channel. No aids to navigation, such as lights to mark the channel, were available. Of the 314 steamboats built from 1811 to 1829 for use on the western rivers, 133 were lost by shipwreck. In spite of that and of the increasingly strong competition of the railways, steamboat navigation continued to increase until about 1880 because all available methods of transportation were required to meet the needs of the rapidly growing population in the valley.

The Ohio in its natural state afforded a poor channel for navigation because of the frequent and long periods of low water. At such times not only was the depth of the channel insufficient but, of greater consequence, there were hazards in the form of bars, rocky reefs, snags, and other obstructions. When the losses in steamboats and cargoes due to channel hazards and the long delays due to low stages became such an economic burden that the very existence of navigation on the river was threatened, the public began to realize the necessity for more adequate channel improvements.

Practically every navigable tributary of the Ohio River lying wholly in one state was initially improved by or with the authority of that state, although practically none of these improvements were properly maintained. Public demand finally caused such tributaries to be placed under federal jurisdiction to the extent required for the maintenance of navigation.

Gallipolis Dam on the Ohio River
A Roller Type with Navigation Locks



LOCK AND DAM NO. 12 AT WHEELING, W.VA.
Typical of Chanoine-Wicket Structures with Bear-Trap Weirs

By Act of May 24, 1824, Congress appropriated $75,000 for the improvement of the Ohio River channel at six sand bars, all below Louisville. These funds were the first noted by Congress for actual channel improvements although the Act of April 14, 1820, provided for an investigation of the river below Louisville.

On interstate waterways, and on other navigable streams where the Federal Government has taken over such rights from the states, all works for improving navigation have been under the supervision of the Corps of Engineers of the U.S. Army. That arrangement was initiated by President Monroe in 1824. It was not until August 30, 1935, however, that an Act of Congress made the arrangement mandatory.

At intervals up to 1844, Congress authorized additional improvements to the Ohio River channel. In addition to dredging at shoals and removal from the channel of snags and other obstructions, timber and stone training dikes were constructed at various bars for the purpose of contracting the channel and increasing scour. Also, at certain localities the channel was improved by the construction of dams across back channels.

After a lapse of 22 years, in 1866 Congress began again to appropriate funds for the construction of dikes, back-channel dams, dredging through bars, and removal of snags and other obstructions. By that time, however, it was realized that such methods could never provide an adequate and dependable channel.

RIVER TRAFFIC AFFECTED BY SERIOUS DELAYS

Commerce on the river was augmented by the transportation of bituminous coal from fields along the Ohio in western Pennsylvania, eastern Ohio, northern West Virginia, and from the southern West Virginia fields along the Kanawha River. These coal shipments could be made only when the natural stage of the river provided sufficient depth for the loaded barges. Coal shipments were frequently suspended for long periods.

For example, in June 1895, when only one lock and dam, No. 1 or Davis Island, near Pittsburgh, had been completed, 1,200,000 tons of coal on 2,500 barges had collected in the Pittsburgh harbor awaiting a rise in the river, which did not come until November 27, a wait of five months. In 1908, the river was too low for coal fleets and packet boats to operate from June 5 to the sixth of the following January, a period of seven months. During most of this period the depth was insufficient to float a skiff over the many shoals in the upper river. The writer saw an automobile being driven across the river bed a short distance below Wellsville, Ohio, about 45 miles below Pittsburgh. A similar crossing was made at Wellsburg, W.Va., about 75 miles below Pittsburgh.

Decadence and possible cessation of transportation on the Ohio River was foreseen unless a more dependable and reliable navigable channel was available. It was apparent also that such a channel could be obtained only by a slack-water system of locks and dams, as the results of the open-channel improvements carried on for many years indicated that no adequate channel depth could be provided by that method. Fixed dams, such as were up to that time the only ones in use in the United States for that purpose, were not practicable.

MOVABLE DAMS INITIATED

After a considerable period of investigation and study, the District Engineer in charge of the Ohio River improvements, in his annual report for 1874, recommended that a lock and dam with Chanoine wickets be constructed across the river about six miles below Pittsburgh. Congress approved the recommendation, and by the Act approved March 3, 1875, appropriated $100,000 for the construction of such a dam as an experiment. Construction was commenced in 1878 but the structures were not ready for operation until 1885. By then two dams of similar type had been constructed in the Kanawha River upstream from Charleston, W.Va., and placed in operation in 1880.

The dam near Pittsburgh was built in four sections—one for the navigable pass with wickets 14 ft long, and three with shorter wickets for regulating discharges through the dam. It was originally planned to operate all the wickets by winches from service bridges above the dam. Before completion of the dam, however, it was decided to operate the longest wickets, those above the pass, from a boat called a maneuver boat. The operation of the shorter weir wickets from a service bridge was found unsatisfactory also. Sudden freshets caused vast quantities of drift to accumulate above the service bridges before enough of the wickets could be lowered to let the drift go through, and a good part of the service bridges were destroyed. A short weir of the hydraulically operated bear-trap type was then installed in 1889. An innovation in this first Ohio River lock and dam was the use of horizontally rolling lock gates instead of the double-leaf, vertically hinged, mitering gates used almost universally in navigation locks before that time. The new-type gates were constructed of pine timber and soon had to be replaced by steel gates.

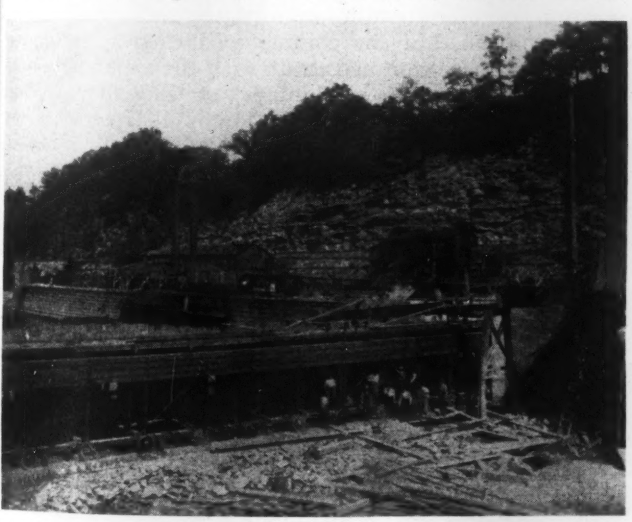
Construction of more locks and dams was subsequently authorized by Congress, but because of inadequate appropriations progress was so slow that the slack-water system was not completed from Pittsburgh to the Pennsylvania state line, only 40 miles below, until 1907. It required nineteen years more to complete it as far down as Cincinnati, but the rate of progress by that time had been so accelerated that in 1929, only three years later, the improvements were completed over the entire river. In no other stream in the world have wicket dams ever been operated in such numbers. These installations are also unique as regards the length of the wickets and the size and length of the river.

Ice Jam at Dam No. 27—Maneuver Boat Lowering Wickets

Until 1909, the project navigable depth was only 6 ft, but in that year, by Act of Congress, it was increased to 9 ft. At that time six of the locks and dams were already completed and three more were so near completion that the necessary modifications for the increased project depth could not be made, and the channel depths in those pools had to be increased by dredging.

The greatest obstacle to navigation on the Ohio River in its natural state was the "Falls of the Ohio" at Louisville, which in reality are a series of rapids with a fall of 26 ft in 3.2 miles at low-water stage. The Falls virtually blocked the river to navigation except at very high stages. By an Act of the Kentucky Legislature of January 12, 1825, a corporation, called the Louisville and Portland Canal Company, was chartered to construct a toll canal around the falls. Of the 6,000 shares of stock issued originally, the Federal Government owned 2,335 shares. By 1855, the Federal Government had acquired all but 5 shares of the stock but did not acquire complete ownership of the canal until some years later when tolls were abolished. The canal was enlarged and new and larger locks were constructed about twenty years ago.

In general, except for improvements in details, the first lock and dam in the Ohio River, completed in 1885, remained the prototype for all later structures in the river, although the last ones were not finished until 1929. On all locks designed after about 1912, two-leaf mitering-type gates were installed, which accounts for the two types of lock gates on the river.

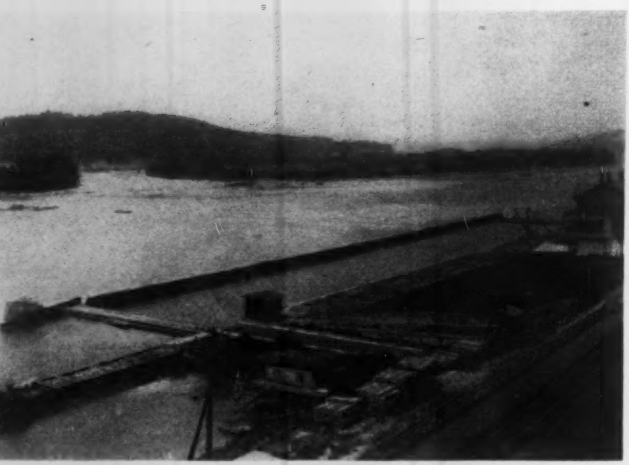


Wooden Rolling Gates Which Were First Installed in Lock No. 1 in 1885—Later Replaced by Similar Gates of Steel

As previously stated, the whole of the original system of slack-water improvements was completed in 1929, but before that was accomplished, the first four of the original locks with movable dams below Pittsburgh had been replaced with two locks with fixed dams of higher lift. That was due to obsolescence and deterioration of the old structures, mostly the former, as deeper and more dependable pools were needed for the immense river traffic in the Pittsburgh area. The first new dam below Pittsburgh was later rebuilt with crest gates of lift type and the normal pool also was raised. Subsequently the next two below the first four of the old locks with moveable dams, Nos. 5 and 6, were replaced by one higher-lift lock and a lift gate type of dam.

Also in recent years, three of the original locks with movable dams between Racine, Ohio, and Huntington, W.Va., have been replaced by one 26-ft lift lock and roller-type dam consisting of horizontal steel cylinders which roll upwards between piers to open the dam partly or fully, and vice versa for closing. It is called the Gallipolis Dam because of its location about 8 miles below the Ohio city of that name. Except for the piers, neither the lift-gate nor the roller type of dam restricts the natural flood discharge capacity of the river. Navigation can never pass over these dams, and they are therefore also called non-navigable types. The Gallipolis roller dam is the largest of its kind in the world.

In theory, all that is necessary to maintain the project depth with Chanoine-wicket dams is to raise and lower the dams as river stages require. In practice, however, it is a rather difficult problem. This was not fully realized until a number of the units in the system were in operation. The narrow limits in height between which the pools must be maintained; the frequent, rapidly changing discharge conditions in the stream; and the very pronounced effect which the manipulation of one dam has on the dams below, are the main factors that complicate operations. Any single dam or any part of the whole series of dams cannot be operated solely on the basis of local conditions, independently of the dams below or, to a lesser extent, of the dams upstream.



First Lock and Dam in the Canalization Project, near Pittsburgh—Replaced in 1921 by a Higher-Lift Structure Downstream near Dam No. 2

All operations must be made with due regard to stream stages, rainfall and weather conditions on the watersheds, and the effect which the operations will have on the other dams. Very close cooperation between the units in the system is necessary. River conditions must be anticipated as much as possible. For that purpose information on river stages, rainfall, and weather over the watersheds is collected daily, and more often when meteorological conditions are changing or are expected to change.

Notice of all maneuvers of dams affecting the dams below must be passed down the river at all times. The usual method of communication is by telephone, augmented by radio, for which transmitters are now located in the District offices in Pittsburgh, Huntington, Cincinnati, Louisville, and Nashville, and in the Division Office in Columbus, Ohio. Telegraph and teletype are also used. Interruptions in communications are a serious matter. Unfortunately they are caused by the same storms which cause rapid changes in the river stages, and thus occur generally when the exchange of information is most necessary. When all communication systems are broken, the operating crews hold themselves ready and act on their own initiative for such quick changes as may be necessary to safeguard the structures in their charge. To maintain a satisfactory navigation project with such a system and under such conditions, requires a supervisory and operating personnel with considerable special knowledge and experience, which is obtainable only by long actual association with the structures.

Drift and ice sometimes cause very serious difficulties in maintaining the pools. Freshets on the watersheds occasionally bring down large quantities of drift which lodges above the locks and dams, blocking the upper approaches to the locks and preventing access to the upper sides of the dams for manipulation of wickets. There have been cases where drift has covered a triangular area extending up the river 900 ft above the axis of the dam on the lock side, the other leg of the triangle reaching entirely across the river, making the total area of the drift approximately ten acres.

Ice is the greatest hazard under which the pools have to be maintained. Fortunately, severe ice conditions do not occur frequently. Ice gorges form above the locks and dams, making it difficult and sometimes impossible for navigation to pass through to enter or leave the locks. Under such conditions, it is difficult and hazardous to open part of the dam to pass the ice through or to lower the whole dam if a rising river requires it.

Now that construction and maintenance of improvements for navigation on the Ohio River have been carried on for a hundred and twenty-five years, what have been the results? Have the large expenditures for the preservation and encouragement of commerce on the river been justified? For many years, or until about the middle of the nineteenth century, the river commerce was almost entirely by what has been generally known as Western Rivers steam-driven packet boats, now almost extinct. Besides freight, these boats carried many passengers, as they were the most convenient means of travel between points along the river.

The packet-boat traffic did not reach its peak until after the Civil War and did not commence to decline appreciably until about 1880, but after that date the decline was rapid. The decline of commerce on the river practically paralleled the decline of the packet boats. By 1905 not many such boats remained, and the freight carried was about 13 million tons, mostly of coal in barges pushed by steam towboats. The commerce reached its low point about 1917, when it was only 5 million tons.

The slack-water improvements then began to be effective. In 1926, when the canalization had been completed from Pittsburgh to Cincinnati, the commerce carried had risen to $19^{1}/_{2}$ million tons, and in 1930, one year after completion over the whole river, it rose to $22^{1}/_{2}$ million tons. In the next two years, on account of the general economic depression, it dropped to about 14 million tons, but in 1932 it began to increase rapidly, although in so doing it was not in harmony with transportation in general, which did not tend to increase until about five years later. Except for a slight decline in 1937 and 1938, the rate of increase was slightly accelerated, so that by 1941, the last year for which data can be given, commerce had increased to $36^{1}/_{2}$ million tons.

Accompanying this substantial and almost continuous increase in the quantity of commerce, was the change in the character of the commodities carried and the increasing length of haul. The greatest increases in tonnage occurred in commodities not previously transported in any great amounts, the principal ones being steel and petroleum products, especially the latter, which in 1941 was the second in tonnage and the first in ton-miles carried.

From the data previously given, it is obvious that navigation improvements on the Ohio have produced satisfactory results. Not only has the river been restored to a position of considerable importance as a medium of transportation, but the commerce it carries has undergone an increase commensurate with the expanded population and industrial development of the valley. While the increase in traffic from the low in 1917 of 5 million tons to the $36^{1}/_{2}$ million tons in 1941 (or a gain of 630%) is strong evidence of the success of the improvements, it is not as accurate a gage as the ton-mileage, which shows a much larger gain.

The ton-mileages of commodities hauled are a more proper index of the transportation services performed than the tons only. The length of the average haul is not available for 1917, but in 1926 it was 55.7 miles. In 1930, the year following the completion of canalization of the whole river, it was 66 miles; and in 1941, 142 miles. The explanation for this gradual lengthening of the average haul is that the commodities showing the largest increases in recent years are those which mostly originate at, or are destined for, points off the Ohio River, and which therefore are hauled long distances on the river. The principal ones are iron and steel, petroleum products, and sulfur.

Coastal Highway Spans Cross Georgia Delta

Fixed Plate-Girder Structures Bridge Altamaha and Darien Rivers

By C. N. Crocker

Bridge Engineer, State Highway Department of Georgia, Atlanta, Ga.

Crossing the Altamaha River Delta, the Coastal Highway encounters four navigable channels. Inadequate structures over the Darien River and the South Altamaha River restricted the movement of heavy traffic. Two high-level plate-girder bridges were therefore planned to replace the old steel-truss swing spans. As reported by Mr. Crocker, the South Altamaha crossing has been completed and the Darien River Bridge will be completed shortly.

For many years the State Highway Department of Georgia has realized the value of that section of the Coastal Highway which extends through Georgia, between South Carolina and Florida, as a medium for handling the heavy vehicular traffic which has been developed by the increase in both freight and tourist transportation. Since 1920, with the valuable assistance of the counties and the Coastal Highway District Association, concerted plans have been in progress to develop this road into a hard-surfaced thoroughfare capable of adequately serving the traveling public. This problem was one to be solved by stage construction in the following steps: (1) grading and draining on reasonable alinement, (2) hard-surfacing for two-lane traffic, and (3) adjusting alinement and planning for four-lane divided paving. Naturally each successive stage would be governed by the development in type and capacity of vehicles to be handled.

FOUR MAJOR BRIDGES INVOLVED

One of the most critical sections, involving major stream crossings, lies within the six miles just south of Darien toward Brunswick. Originally a railroad provided transportation, but this was abandoned, and in 1919–1920 the State Highway Department of Georgia took over the fills and structures and reconstructed to accommodate a highway. There were four major bridges involved—over the Darien River, Butler River, Champney River, and South Altamaha River, all outlets of the Altamaha River as it passes through its delta on its way to the Atlantic Ocean. The first stage of construction made use of the existing railroad bridges, which were reconstructed of untreated cypress and pine timber. Some years ago the structures over the Butler and Champney rivers were replaced with modern H-20 structures of 24-ft roadway, with provisions for similar parallel bridges whenever the traffic justified four-lane construction.

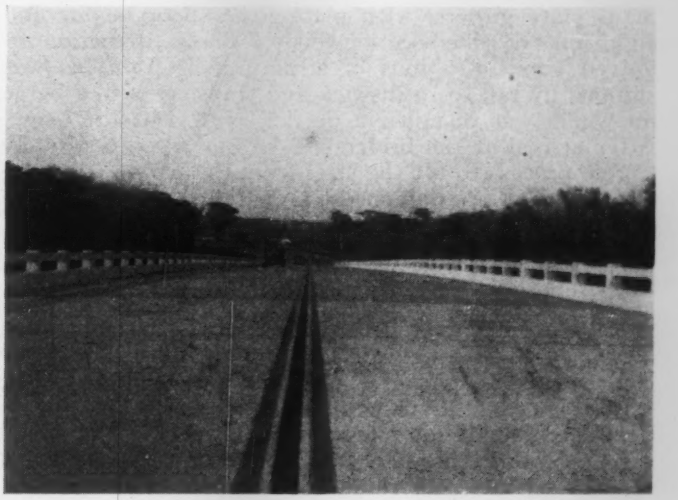
The next sequence in the stage planning was the provision of adequate structures over the South Altamaha and Darien rivers. A general description of the construction problems involved at these two rivers is necessarily divided into two parts—bridge construction and grading operations.

At both the Darien and South Altamaha rivers, the main spans as constructed by the railroad and subsequently adapted to highway use, consisted of one 240-ft hand-operated, steel-truss swing span on concrete piers, providing a roadway clearance of only 14 ft. The untreated cypress and pine-timber trestle approaches provided only a 15-ft roadway. This construction at first served traffic adequately but later conditions became such as to endanger the traveling public both from the viewpoint of roadway width and capacity.

SOUTH ALTAMAHA RIVER BRIDGE

On February 9, 1940, the State Highway Department let the first contracts for the construction of the South Altamaha River crossing, covering approach embankments, temporary paving, and the reinforced concrete part of the bridge itself. The contract for the structural steel was let March 28, 1941. The new bridge was located about 500 ft upstream from the old structure, requiring approximately 5,000 ft of relocated length of roadway.

The new bridge has a total length of 2,382 ft, provides a 40-ft roadway between curbs (with a 10-in. offset to rail



Crossing the South Altamaha River on the Coastal Highway, Drivers Formerly Used Plank-Floored Remodeled Railroad Bridge (Left) Now Superseded by Spacious Deck (Right)

STABLE MATERIAL BEING PUMPED INTO PLACE

posts), and was designed for H-20 loading. Approach grades of 3.75% are connected at the vertex with a 1,000-ft vertical curve, raising the bridge floor to an elevation of 58.5 ft above mean low water. The main piers and 1,548 ft of bridge approach are of reinforced concrete. The concrete river piers support structural steel deck-plate girders of variable section, providing an 80-ft horizontal clearance between treated timber fenders for navigation. There is no movable span, but the vertical clearance of 50 ft above mean low water was approved by the War Department as sufficient to permit the passage of river-borne traffic.

The structural-steel river spans consist of two 85-ft deck plate girders, and two series of 91–150–91-ft deck plate girders of the cantilever and suspended-span type, the total length being 834 ft. The end girders extended over the 91-ft anchor spans and were cantilevered for a distance of 32 ft beyond the second pier. The 86-ft suspended span was supported on the cantilever arms and was fastened by means of eyebar hangers with pin connections.

In general, the structural steel design consisted of two main plate girders, with plate-girder floor beams and eight lines of rolled-section joists. The construction involved about 875 tons of structural steel. This was shipped by rail to Brunswick and transferred to a barge on which was installed a derrick with a 100-ft boom. After arrival at the bridge site, it was a comparatively simple process to lift the steel from the barge and place the girders in position on the prepared pier caps and the other members in their respective locations. This procedure entailed very little handling at the site.

UNTREATED PILES USED

Reinforced concrete river piers consist of two battered columns tied with web walls extending approximately 10 ft above mean high water. The individual footings were carried about 20 ft below the river bed and supported on untreated foundation piles driven into coarse compacted sand. Within the steel sheet-piling cofferdams, 15-ft seals were poured before the cofferdams were pumped out to permit pouring of the remaining 5-ft thickness of the base. Considerable trouble was encountered in driving the piles on account of the hard consistency of the compacted sand, which contained some shell and was almost a shell-sand marl.

Approaches consist of thirty-six 43-ft reinforced-concrete deck-girder spans supported on three-column reinforced-concrete bents, with bottoms of footings about 13 ft below marsh level. The columns were battered and tied together with concrete struts. Untreated foundation piles were driven into hard compacted sand. The muck overlying the marsh varied from 14 to 27 ft in depth. The top area was heavily matted with cypress and pine roots and marsh vegetation which extended down from 2 to 4 ft.

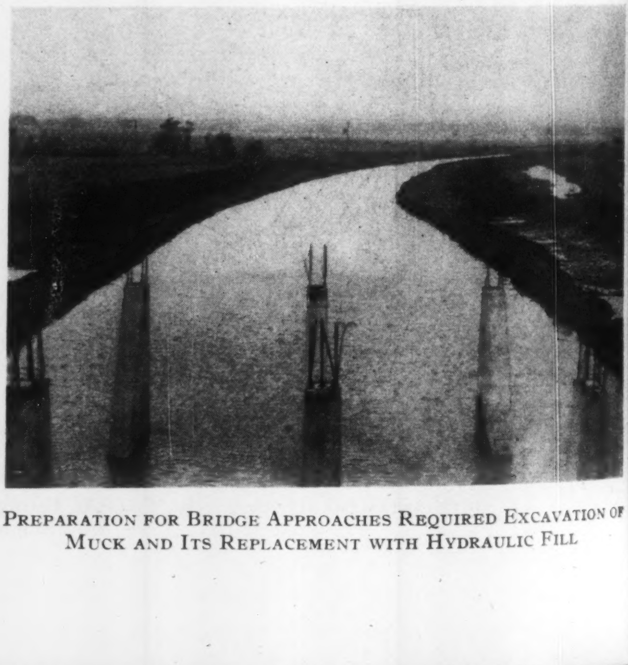
The underlying muck varies in consistency from almost fluid to semistable, and is impregnated with fine sand, which affords lateral support for the foundation piles. At the bottom of this muck there is a coarse compacted sand, into which, after hard driving, the foundation piles penetrated about 10 ft. Steel sheet piles 20 ft long were driven to form cofferdams for the footings, and after the piles were placed the interior of the cofferdams was dried and the footings poured. Occasional boils and blow-ins occurred, but very little trouble was encountered.

This job is under construction at the present time and will be completed in the fall of 1944. The contractor has a well-organized construction setup with the following major equipment:

NO.	EQUIPMENT	NO.	EQUIPMENT
1	steam-powered stiff-leg derrick with 90-ft boom, mounted on a lighter 36 by 72 ft	4	4-in. centrifugal pumps
		2	2-in. centrifugal pumps
		1	6-in. centrifugal pump
2	$^5/_8$-cu yd draglines	4	4-in. double-diaphragm pumps
2	$^3/_4$-cu yd draglines	3	2-in. force pumps
1	1-cu yd dragline	4	concrete vibrators
3	14s concrete mixers	1	No. 3 steam hammer
1	10s concrete mixer	1	No. 1 steam hammer
1	batching plant	1	No. 5,000 steam hammer
1	32-ft tugboat	2	No. 400A steam pile extractors
3	3-in. centrifugal pumps	4	2,500-lb drop hammers

These major units are adequately supplemented by small tools and equipment.

The contractor's equipment and plant layout were well adapted to handle all the problems of construction. On



PREPARATION FOR BRIDGE APPROACHES REQUIRED EXCAVATION OF MUCK AND ITS REPLACEMENT WITH HYDRAULIC FILL

Completed South Altamaha Crossing

account of the character of the marsh and the limited storage space, it was necessary to drive pile-trestle runways leading from the paved road to the bridge site. A central mixing plant—complete with one 30-ton bin, one 1-cu yd batcher, one mixer, and an elevator—was installed on a pile platform adjacent to the bridge. Lateral trestle runways were constructed on which a narrow-gage, gas-operated tram moved materials. One steam power hammer, three vibrators, and three draglines were supplemented by other miscellaneous equipment. It was necessary to transport the majority of the equipment and materials on a company-owned barge.

On the nine river piers the contractor used a steam-powered, steel stiff-leg derrick, which was mounted on a lighter 36 by 72 ft. This derrick, which had a rotating boom 90 ft in length, handled all the foundation work on the nine piers, which included assembling and dismantling the steel sheet-pile cofferdams, excavating footings with a 1-cu yd clamshell bucket, driving the foundation piles, and pouring the seal footings. A dragline mounted on a lighter 16 by 32 ft followed the foundation work, handling forms, reinforcing steel, and concrete. Delivery of structural steel was delayed considerably on account of priorities on steel plates, and actual construction was interrupted until this material was made available.

GRADING OPERATIONS

The approach fills to the South Altamaha Bridge extended 1,050 ft on the north end and 1,514 ft on the south end. At both ends the new location tied into the existing embankment and it was necessary to construct temporary detour bridges to handle traffic. On account of the marsh condition, it was necessary to remove the muck before the hydraulic fill was placed in order to reduce the danger of excessive subsidence. The fill varied in height from 3 to 23 ft above the original ground level and provided a 52-ft shoulder width, with design slopes of approximately 4:1. The muck was removed by draglines for a depth of about 6 ft, except adjacent to the bridge ends, where the depth was about 12 ft.

Piers for Concrete Approach Spans

At one time there were six draglines operating to expedite this work. During these operations, considerable difficulty was caused by the sliding in of the edges of the excavation due to the weight of the dragline. It was therefore found expedient to stabilize these edges by driving a line of timber piling some 10 ft away from the excavation. The muck was cast to the edge of the right of way to be later thrown back on the slopes of the fill to provide satisfactory material in which to grow grass. The fill was placed through a 16-in. shore pipe leading from a dredge placed about 1,000 ft upstream from the new bridge. This required 3,000 ft of pipe, of which about 700 ft was on pontoons. The dredge was of wooden hull construction, 120 by 32 ft, equipped with wood spuds 45 ft long. The power plant was a diesel engine of 800 hp used for direct drive for the main pump at 240 rpm. There was also a diesel engine of 450 hp, driving a 175-kw generator at 730 amp and 120–240 volts at 300 rpm, to supply power for the cutter motor.

Material for the hydraulic fill came from a depth of as much as 25 ft below the river bed and was a coarse sand containing very little organic matter. This was ideal for the embankment. Hauled-in borrow material was used in lieu of the hydraulic fill in some places. Sheepsfoot rollers were utilized where necessary to obtain compaction. The surfacing consists of a single-surface treatment laid on a 6-in. compacted bituminous stabilized roadbed. This temporary paving was provided to serve until major subsidence of the fills occurs, when it is anticipated that a permanent paving will be placed. This project was opened to traffic on November 4, 1943.

DARIEN RIVER BRIDGE

On November 10 1942, the State Highway Department let contracts for the construction of the bridge over the Darien River, covering all concrete and steel portions of the bridge, the

STEEL FOR DARIEN RIVER CROSSING WAS ERECTED BY DERRICK SCOW

approach fills, and the temporary paving. The new bridge will be 1,452.5 ft long and will have a center curb dividing the roadway into two 22-ft lanes. There will be a 4-ft sidewalk on each side of the roadway.

One of the main difficulties in the design was to provide satisfactory vertical clearance for navigation and still meet the intersection street grades in the City of Darien. It was greatly desired to use a fixed span so as not to hamper the movement of either the highway or the river traffic. This was accomplished by using a 3.50% tangent grade on the Darien end and a 2.50% tangent grade on the opposite end, connected with a 400-ft vertical curve.

It was necessary to provide a restricted depth from the top of the roadway to the bottom of the main girders. This was done by using three main girders of variable section, allowing the center girder to project above the finished roadway elevation. Curbs, 10 in. high, battered 4 in., were placed on each side of the center girder, an arrangement which supplied the raised center dividing curb previously mentioned. By this means a vertical clearance of 37.13 ft was obtained from mean low water to the bottom of the girders at a point over the fender system of treated timber piles. A horizontal clearance of 110 ft is provided for navigation.

The south approach consists of twenty-two 40-ft reinforced-concrete girder spans. The main river spans consist of one series of 91–150–91-ft deck plate girders of the cantilever and suspended-span type, and one simple deck-plate-girder span of 80 ft. The north approach is made up of four 40-ft reinforced-concrete deck girder spans, similar to the south approach. A center dividing curb was provided on the concrete decks to correspond with that on the structural steel spans.

FOUNDATIONS AND PIERS

Both end bents are composed of ten 15-in. pre-cast concrete piles with a concrete cap. The approach decks are supported by three-column concrete bents on untreated timber foundation piles. The columns are battered $^1/_4$ in. in 12 in. in both directions and tied with a horizontal concrete strut. The bottoms of the caps were curved for the sake of appearance. The main river piers are similar to the approach bents, except that they are of larger section, and the columns are tied together with a web wall 15 ft deep and 18 in. thick. On account of the depth of the water, concrete seals were necessary around th foundation piles.

The new structure is substantially at the same site a the old bridge, and three old concrete piers were remove to the bed of the stream. One of these was a roun pivot pier, the removal of which caused considerabl difficulty.

In general, the design of the structural-steel main span consists of three variable-section plate girders with 33 in. wide flange floor beams and ten 16-in. rolled-beam joists. In order to fit the roadway grade, some of th joists were framed into the floor beams, while others wer supported on top of them. Approximately 580 tons o structural steel were required.

GRADING OF APPROACHES

The approach fill on the north end consists of a short ramp just sufficient to tie into the present grade at a street intersection in the City of Darien. The south end however extends 1,260 ft before it ties in with the present fill. Construction conditions are similar to those at the South Altamaha River crossing, and it was necessary to remove approximately 8 ft of muck before placing the hydraulic embankment. The same methods of construction and equipment are to be used as previously described for the South Altamaha River Bridge. The fill at present is about 90% completed.

The projects were divided into several contracts and the following tabulation gives information as to costs and contractors:

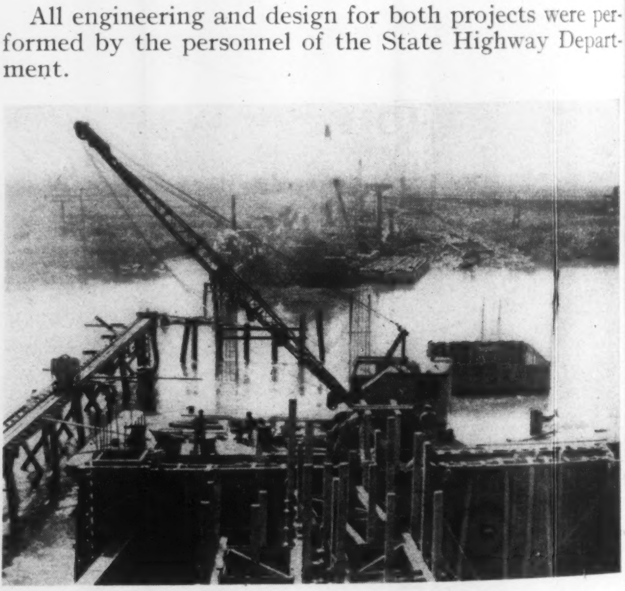
SOUTH ALTAMAHA

Bridge items (except structural steel)	W. F. Scott Co., Thomasville, Ga	$457,772.04
Structural steel (furnished and erected)	Nashville Bridge Co., Nashville, Tenn.	138,764.00
Grading and paving	Scott Construction Co., Thomasville, Ga.	207,607.89
Total		$804,143.93

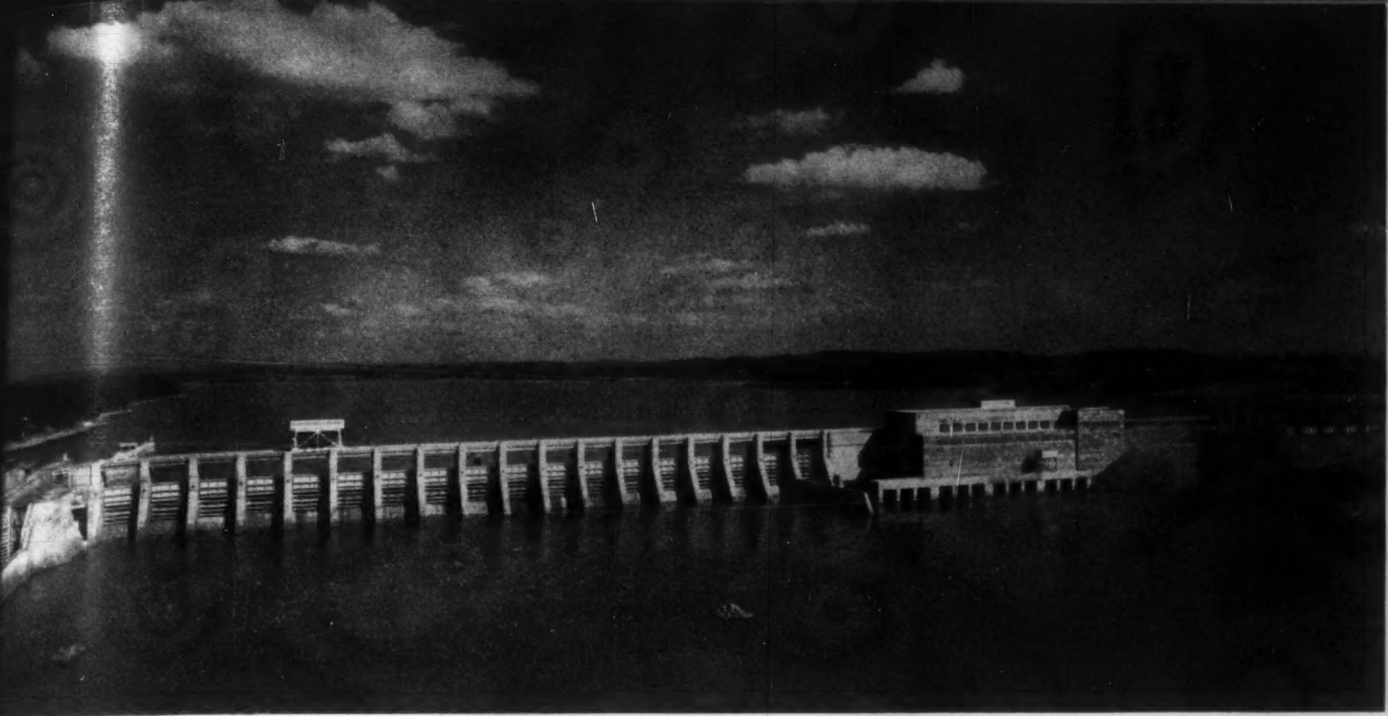
DARIEN RIVER

Bridge items (except structural steel)	Scott Construction Co., Thomasville, Ga.	$837,150.00
Structural steel (furnished and erected)	Nashville Bridge Co., Nashville, Tenn.	98,940.00
Grading and paving	Scott Construction Co., Thomasville, Ga.	162,072.96
Total		$1,098,162.96

All engineering and design for both projects were performed by the personnel of the State Highway Department.



DURING CONSTRUCTION OF RIVER PIERS FOR DARIEN RIVER BRIDGE, NARROW-GAGE RAILWAY (LEFT) SUPPLIED MATERIAL



Chickamauga Project—Typical of Main-River Low-Head Storage Developments

Basic Hydraulics of Water Storage Projects

I. Project Layout and Lock Filling Systems

By George R. Rich, M. Am. Soc. C.E.

Chief Design Engineer, Tennessee Valley Authority, Knoxville, Tenn.

In reviewing the broad principles which have been most serviceable to TVA engineers in their designs, Mr. Rich has had reference to both American and European literature of the past twenty years. As an introductory principle, he mentions the habit formed by engineers of going back to basic physical actions and proceeding from that point to create a design with the best modern tools available. This article is the first part of a paper presented before the Hydraulics Division of the Society.

ESTABLISHMENT of design principles and standard designs has proved to be a great time and money saver for the Tennessee Valley Authority. Design standards are not unusual, as most engineering organizations have used them extensively. Standard designs on the other hand have seldom been employed so widely on projects the size of the hydroelectric development in the Tennessee Valley. A series of articles on the design of recent TVA projects appeared in Civil Engineering from March to October 1943.

The advantage of carefully designing a tainter gate, for instance, and using the same design for repeated installations, can readily be seen. This expedient, desirable though it may be, is only possible if a sound design has been worked out in the first instance and if conditions of installation can be controlled so that they are identical in every case. Both these requisites were supplied by the TVA. Skillful designers worked out the original designs, giving attention to most minute details. Then, since authority for the development of many projects was centralized, it was possible to duplicate conditions at a number of installations.

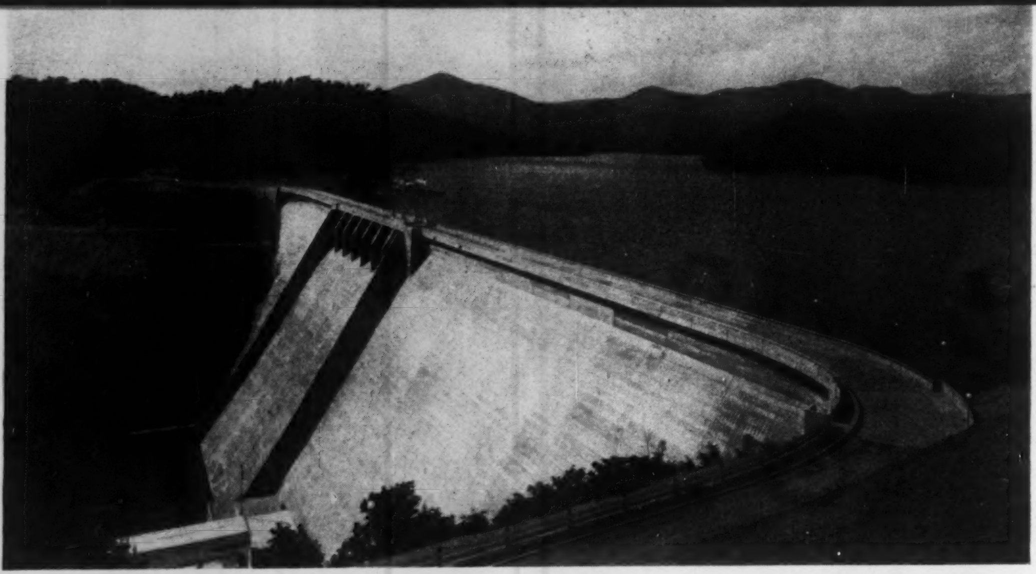
With respect to layout, as well as to location and mode of operation, the hydraulic projects of the Authority can be classified according to three main divisions: low-head main-river projects, main-tributary storage projects, and secondary-tributary storage developments.

Typical of the arrangement of structures for a low-head development is the Chickamauga project. As a general rule, the power station, with its high load concentrations and required freedom from unequal settlements to insure satisfactory operation of the generating machinery, is placed on the best foundation rock available at the site. It is also essential that the location selected afford direct transfer from the station crane to railroad cars and trucks. To obtain the requisite economy in length of generator leads and operating cables, and as a practical matter of ready accessibility from an operating standpoint, the switchyard and power station must naturally be grouped as a single unit.

The navigation lock, operated by the Corps of Engineers, is generally located on the opposite bank from the power station, thus preventing any possible conflict in operation and maintenance between the navigation and power generation functions. A long upstream approach wall and downstream guard wall are provided to permit tows to enter the lock at the requisite reduced speed without hazard or interference from the spillway.

The spillway is placed in the central part of the stream, the most advantageous position for distributing the discharge uniformly over the stream bed so as to minimize the effects of erosion. It frequently happens that this is also in the zone where the highest natural foundation rock occurs, an important consideration for most economical apron construction. At low-head main-river projects, the operating program does not contemplate reduction of reservoir levels below the crest of the spillway, and consequently the entire regulation is accomplished by crest gates.

In the layout of higher-head structures for a typical main-tributary storage project, Hiwassee for example, the dominant consideration is the economical location of



HIWASSEE DAM PROVIDES A HIGH HEAD WITH REGULATION OF TRIBUTARY WATERS

the main dam to fit the natural bedrock topography and deliver the comparatively high-velocity discharge to the natural river channel with the least disturbance. In the normal drawdown cycle, the reservoir level drops far below the crest of the dam, and any subsequent regulation must be effected by means of low-level sluices. Crest gates of several types are provided for the control of flood discharges.

The Apalachia project is typical of the arrangement of structures for developments on tributary streams with steep slopes. The impounding structure is an overflow gravity dam equipped with tainter gates and connected to the power station by a long concrete-lined tunnel so as to afford development of head to the limit economically justified. A surge tank of conventional design is excavated in the solid rock just upstream of the power station. The steel penstocks are of modern ring-girder design, controlled by butterfly valves.

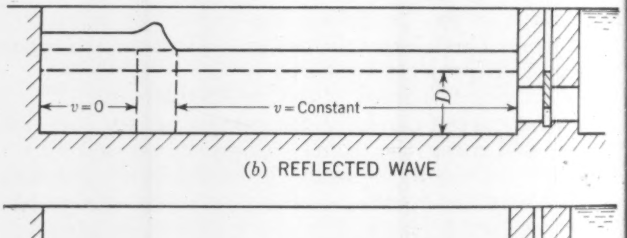
With this brief perspective of typical projects as a background, a more detailed consideration of major individual structures will now be given. First comes the question of filling systems for navigation locks.

FILLING SYSTEMS FOR NAVIGATION LOCKS

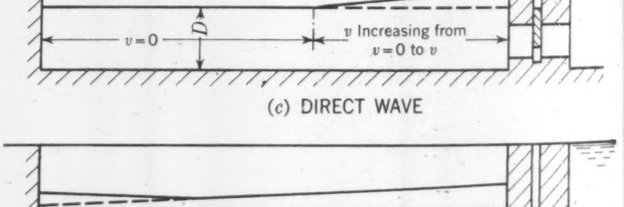
In designing such systems, the problem is to reconcile two conflicting requirements: a reasonably short filling time (10 or 15 minutes), so as not to penalize traffic; and minimum disturbance to the shipping in the lock so as to avoid hazard either to vessels or to lock gates. With the exception of a few rough calculations for the preparation of model designs for testing, the practical problem cannot be solved by extended computations, but must be handled in the laboratory. Nevertheless, it is unfair to infer that analytical studies have no value in lock design. By isolating certain of the major tendencies and studying them singly in simplified form, we have been able to obtain many distinct aids to better planning and interpretation of model tests.

One of the most fundamental factors is the simplified lock chamber of Fig. 1. The rectangular valve is opened instantaneously, and in accordance with the familiar equation for waves of small crest height in proportion to channel depth, a wave is propagated downstream, changing the velocity in the chamber from zero to a constant value, which is readily calculated. On arrival at the downstream gate, this wave is reflected and travels back upstream, reducing the chamber velocity to zero. This cycle, which can easily be observed in models and prototypes, is repeated until levels are equalized. If a vessel were placed in such a chamber, it would be driven violently first in the direction of the lower gate and then in the opposite direction by a force corresponding to the difference in hydrostatic head produced by the superelevation due to the wave crest. The essential feature to note is that the motion of the ship is a characteristic longitudinal shuttling action, first in the direction of the lower gate and then in the direction of the upper gate. In the present case the impulse effect due to jet action from the discharge in the vicinity of the valve may be assumed to produce only secondary effects. The effect of imparting a slow uniform motion to the valve, instead of the initial instantaneous action, is illustrated in Fig. 1. Instead of having the bore effect of a vertical-front wave, the water surface is inclined at a slope, as can readily be shown by conventional hydraulic computations. The flat slope continues until the arrival of a reflection from the lower lock gate.

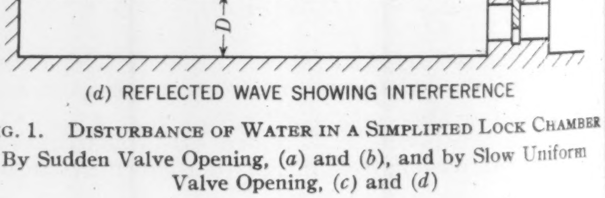
On the basis of the foregoing discussion, it is evident from intuition that the unbalanced hydrostatic force acting on the vessel, and responsible for its characteristic longitudinal shuttling motion, is dependent not on the chamber velocity but only on its acceleration. Consequently the most effective means of reducing the disturbance to the ship is to make the initial valve opening very small and allow the discharge to increase at a uniform rate. Similarly, in the final stages of lockage, the inflow should decrease slowly at a uniform rate. By means of Fig. 2, this vital principle can be substantiated.



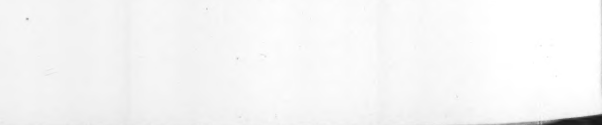
(a) DIRECT WAVE



(b) REFLECTED WAVE



(c) DIRECT WAVE



(d) REFLECTED WAVE SHOWING INTERFERENCE

FIG. 1. DISTURBANCE OF WATER IN A SIMPLIFIED LOCK CHAMBER
By Sudden Valve Opening, (a) and (b), and by Slow Uniform Valve Opening, (c) and (d)

Referring to Fig. 2 (*a*),

Momentum equation, $Ft = M(V - O)$

$$\frac{w}{2}[(D + Z)^2 - D^2] \times 1 = \frac{w}{g}CV(D + Z)$$

Hydraulic continuity, $CZ = (D + Z)\,V$ or $Z = \frac{DV}{C}$

Wave velocity, $C = \sqrt{gD}$

Referring to Fig. 2 (*b*),

Unbalanced force on ship $= \frac{wb}{2}(D^2 + 2DJL + \overline{JL}^2 - D^2)$

Displacement $= wLBD = K$

Unbalanced force $= KJ = \left(\frac{K}{gBD}\right)\frac{dQ}{dt}$

Referring to Fig. 2 (*c*),

Vertical-front wave, $Z = \frac{(D + Z)V}{\sqrt{gD}}$ or $\frac{DV}{\sqrt{gD}}$

Sloping-front wave, $dZ - \frac{D}{\sqrt{gD}}dV$

Slope $J = \frac{dZ}{dt}\frac{1}{\sqrt{gD}} = \frac{1}{g}\frac{dV}{dt}$

Discharge $Q = BDV$, $dV = \frac{dQ}{BD}$; Slope $J = \frac{1}{gBD}\frac{dQ}{dt}$

The derivation of the expression for wave velocity in the chamber is taken bodily from the familiar equation in channel hydraulics for waves of small crest height in proportion to stream depth, while the simple derivations for the slope and unbalanced force on the ship are readily apparent from the diagrams.

At first thought it might appear that observance of this principle would penalize traffic by markedly increasing the filling time. Bearing in mind that in actual operation no type of valve is instantaneous in its action, it will be appreciated that since much larger conduits and valves can be employed, the total time may actually be decreased. This is due to the fact that, while the initial discharge is very small, the discharge in later stages under falling head is very much increased. Only the acceleration, not the chamber velocity, must be kept small. A useful rule in making comparisons is that the total filling time is the sum of the time that would be required for instantaneous opening plus one-half the time required for valve operation.

The presence of a ship of any appreciable size in the lock chamber will naturally modify the elementary action. Not only is the primary wave subdivided and reflected, but in addition the pitching action of the ship is responsible for the propagation and interference of secondary waves, which can be predicted by tedious calculation only in the early stages. Nevertheless, the parent action originally outlined does bear a very intimate relation to the phenomena observed in model tests and prototype lockage.

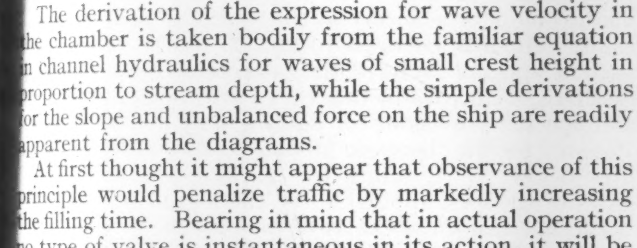
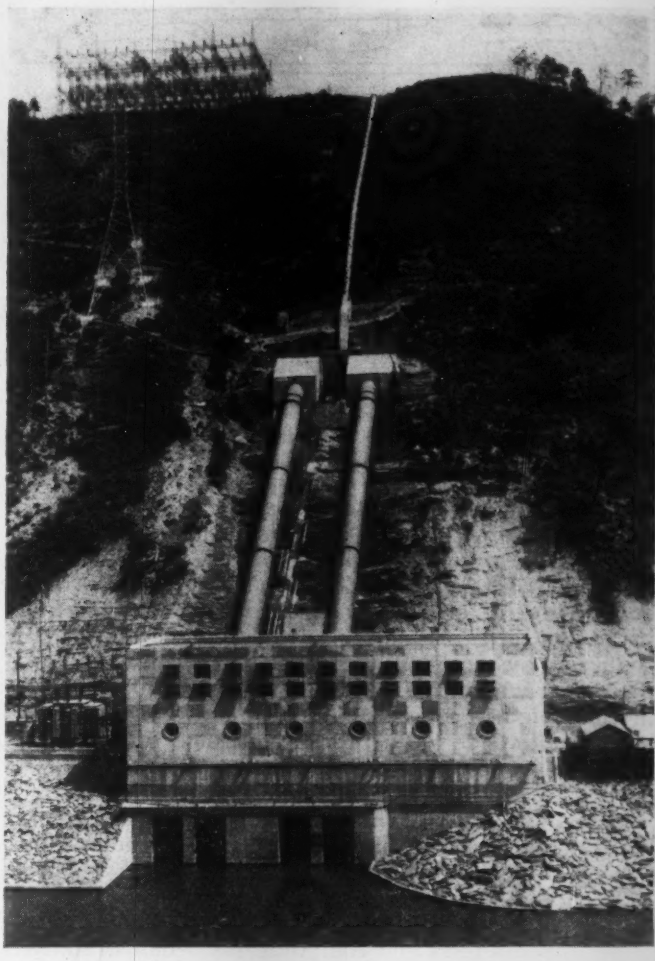


FIG. 2. (*a*) WAVE CREATED IN A LOCK CHAMBER, AND (*b*) SLOPE OF WATER SURFACE ON A SHIP IN THE LOCK



APALACHIA PROJECT, ON A TRIBUTARY WITH STEEP SLOPES

An 8-Mile Tunnel Connects Spillway and Power House

One type of lock design embodying these principles, and used very extensively in Europe for lifts as high as 35 ft, is commonly known as the loop system (Fig. 3). The filling conduits are located entirely in the headworks section, adjacent to the lock gates, and are frequently controlled by a flat rectangular fixed-roller lock valve operating with slow uniform vertical velocity in combination with an orifice of the type shown. The initial valve opening is obviously very small; and the cross-sectional area, and consequently the inflowing discharge, always increases, and in later stages decreases, very gradually. As a result the water surface in the lock chamber has a flat slope and the interference from the action of direct and reflected waves is most favorable.

A very advantageous accessory feature is to employ the venturi principle to permit enlarging the conduits between the valve and the point of discharge and entrance, materially reducing entrance losses and impulse effects by decreasing the velocity at the point where the conduit enters the lock chamber. Baffles are sometimes used, depending upon the lift. For lifts below 25 ft, the only provision commonly used is a diverging conduit.

The venturi-loop type of design was investigated very extensively in connection with the Kentucky lock of the Authority, but the lift was so high (about 70 ft) that the

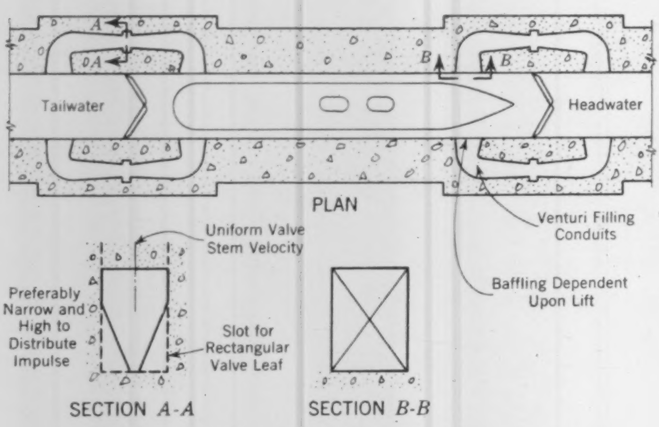

Tailwater
Headwater
A
A
B
B
PLAN
Venturi Filling Conduits
Baffling Dependent Upon Lift
Uniform Valve Stem Velocity
Preferably Narrow and High to Distribute Impulse
Slot for Rectangular Valve Leaf
SECTION A-A
SECTION B-B

FIG. 3. A LOOP-CONDUIT LOCK FILLING SYSTEM

impulse effects due to the discharge velocity required extensive baffling. Model tests incorporating the baffling provisions showed very quiet and satisfactory lockage. Also, comparative cost estimates actually showed a saving in favor of this type of design over all other alternates, but the cost differential was not considered sufficient to justify pioneering so far in advance of the lift of 35 ft confirmed by successful prototype operation.

In this country, the most familiar type of filling system in common use consists of main longitudinal conduits located in the side walls of the lock chamber and discharging into the chamber by means of numerous short lateral ports (Fig. 4). Historically, this type of design was possibly initiated by cut-and-try methods before any appreciable research work was available on the action of lockage. In all but the most recent designs, the ports were equally spaced with a view to distributing the incoming discharge uniformly over the entire lock chamber.

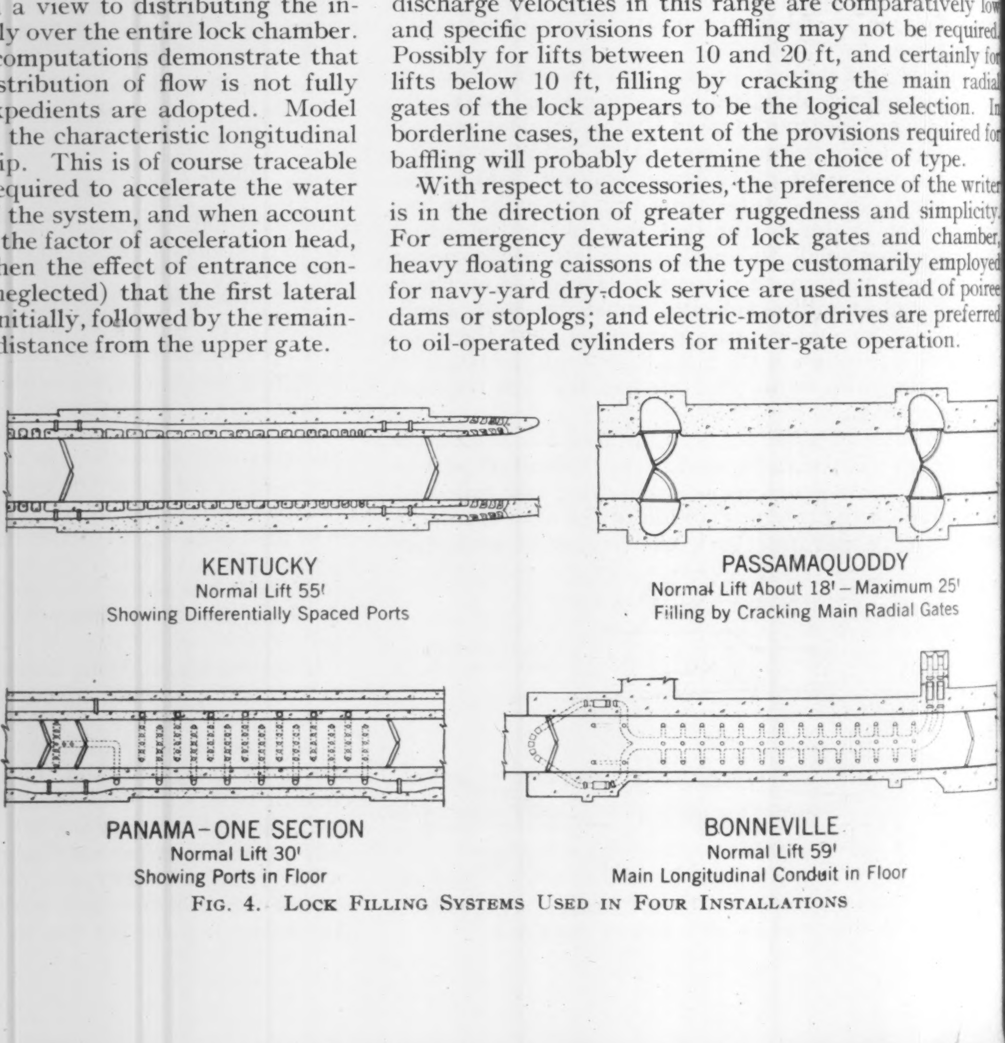
Both model tests and computations demonstrate that the intended uniform distribution of flow is not fully attained unless special expedients are adopted. Model tests for this type exhibit the characteristic longitudinal shuttling action of the ship. This is of course traceable to the fact that time is required to accelerate the water in the various branches of the system, and when account is taken in the analysis of the factor of acceleration head, it is at once apparent (when the effect of entrance contractions in the ports is neglected) that the first lateral port will come into action initially, followed by the remaining ports in order of their distance from the upper gate.

In prototype action it is frequently observed that not the first, but occasionally the last, or some intermediate port, gives the maximum initial discharge. As has already been indicated, this is due to the effect of differences in contraction effects at the entrance to the ports and can be predicted analytically. Model tests of the Watts Bar lock indicate that in early stages of filling, when the depth of water in the chamber is comparatively low, flow in the first port adjacent to the upper lock gates will actually be reversed, owing to the ejector action of th high velocity of flow in the main longitudinal condui The important point to be emphasized in connectio with this type is that the simple expedient of equall spaced numerous ports does not in itself insure uni formity of discharge. Frequently this must be obtaine by comparatively slow opening of the lock valves and b differential spacing of the lateral ports. As a matter o practical design, this is done by reference to past ex perience and laboratory adjustments of port spacing.

For lock installations in which the lift is 20 ft or less consideration should certainly be given to the expedien of filling the chamber by cracking and slow initial openin of the main radial gates of the lock. The mechanics o operation of this design is essentially that of the simplified lock. As would naturally be expected for lifts in th higher brackets, some form of baffling arrangement located on the gates is necessary to absorb the energy of th issuing jet and thus reduce impulse effects in the chamber

In the present state of development of the art of lock design, the most logical choice of filling system for lift above 35 ft appears to be the longitudinal conduit system with differentially spaced lateral ports, operated i conjunction with a relatively slow-speed valve of larg cross-sectional area. Although conventional designs up to the present time seldom include the V-port feature illustrated in Fig. 3, it is the opinion of the writer tha this might advantageously replace the radial valves adopted for many installations. On large and importan developments, it is recommended that model tests and comparative estimates be prepared for an alternative venturi-loop type incorporating suitable baffling provisions. For lifts between 10 and 35 ft, the venturi-loop system appears to have the economic advantage; the discharge velocities in this range are comparatively low and specific provisions for baffling may not be required. Possibly for lifts between 10 and 20 ft, and certainly for lifts below 10 ft, filling by cracking the main radial gates of the lock appears to be the logical selection. In borderline cases, the extent of the provisions required for baffling will probably determine the choice of type.

With respect to accessories, the preference of the writer is in the direction of greater ruggedness and simplicity. For emergency dewatering of lock gates and chamber, heavy floating caissons of the type customarily employed for navy-yard dry-dock service are used instead of poiree dams or stoplogs; and electric-motor drives are preferred to oil-operated cylinders for miter-gate operation.


KENTUCKY
Normal Lift 55'
Showing Differentially Spaced Ports

PASSAMAQUODDY
Normal Lift About 18' – Maximum 25'
Filling by Cracking Main Radial Gates

PANAMA–ONE SECTION
Normal Lift 30'
Showing Ports in Floor

BONNEVILLE
Normal Lift 59'
Main Longitudinal Conduit in Floor

FIG. 4. LOCK FILLING SYSTEMS USED IN FOUR INSTALLATIONS

Monolithic Concrete Seaplane Hangars

FACED with a shortage of critical materials, designers of seaplane hangars for an East Coast naval air station chose reinforced concrete. The resulting structures exemplify the marked progress that has been made in the use of this material. Unusual problems were encountered in the design, and the framing system which resulted was unique. Through the use of rolling scaffolding the hangars were built quickly with economy of materials. A discussion of general problems and their solution is presented by Messrs. Zaborowski and Gruenwald, followed by a detailed account of the moment distribution within the hangar framing.

Unique Design Used at East Coast Naval Air Station

By Robert Zaborowski, Jun. Am. Soc. C.E., *and* Otto Gruenwald,

Structural Engineers, Roberts and Schaefer Company, Chicago, Ill.

DEMONSTRATING the adaptability of reinforced concrete to structures with long roof spans, six seaplane hangars for the Navy have been erected t one station in the East. Plan dimensions of one double eaplane hangar are 413 by 250 ft. Each hangar has a ear door opening of 160 ft, separated and flanked by two-tory leantos 31 ft wide. The curved roof of the hangar a thin shell of reinforced concrete that is supported by rches spaced 35 ft 6 in. on centers. The shell design utiles the Z-D system by Roberts and Schaefer Company. rches are made monolithic with the main bents of the wo-story leantos. While there is no expansion joint n the transverse direction, the building is divided longiudinally into two 71-ft units and two 54-ft 3-in. units. These divisions, together with the use of construction oints, enabled the contractor to pour the leantos ahead f the hangar roofs. The design permitted each hangar oof section to be poured independently of the adjacent ection. In this way a cycle of operations was possible nd the greatest construction speed realized.

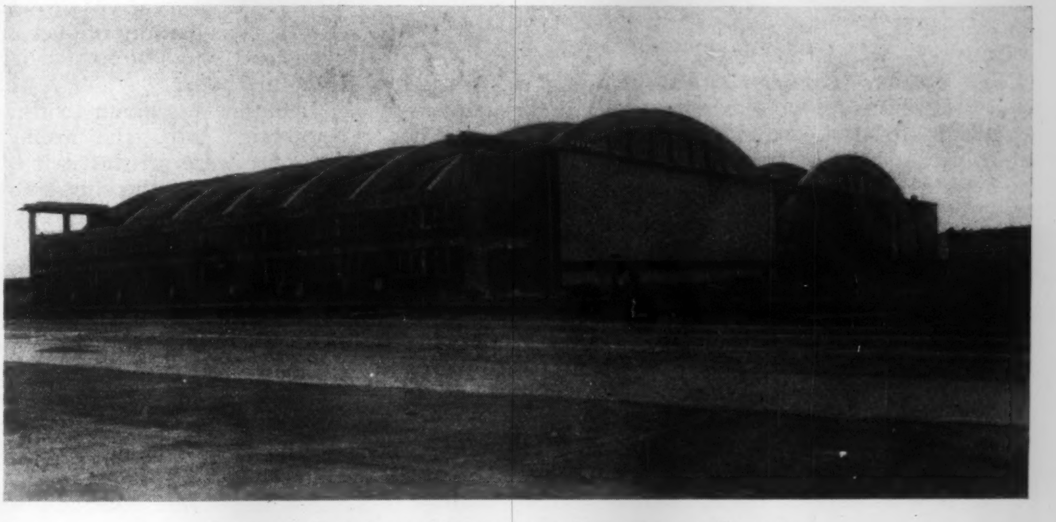
INTERMEDIATE STIFFENING RIBS

Additional transverse stiffness is provided for the shell lab (nominally $3^1/_2$ in. thick) by intermediate stiffening ibs 10 by 20 in., located midway between arches. Beides increasing the buckling safety of the shell, the small ibs distribute concentrated loads into the shell. The rch springing line is 28 ft above the finished floor level nd the high point of the ceiling is 55 ft above this floor level. Reinforced-concrete door girders provide for doors 40 ft high at each end of the hangars. Conventional beam-and-girder framing was used for the leantos.

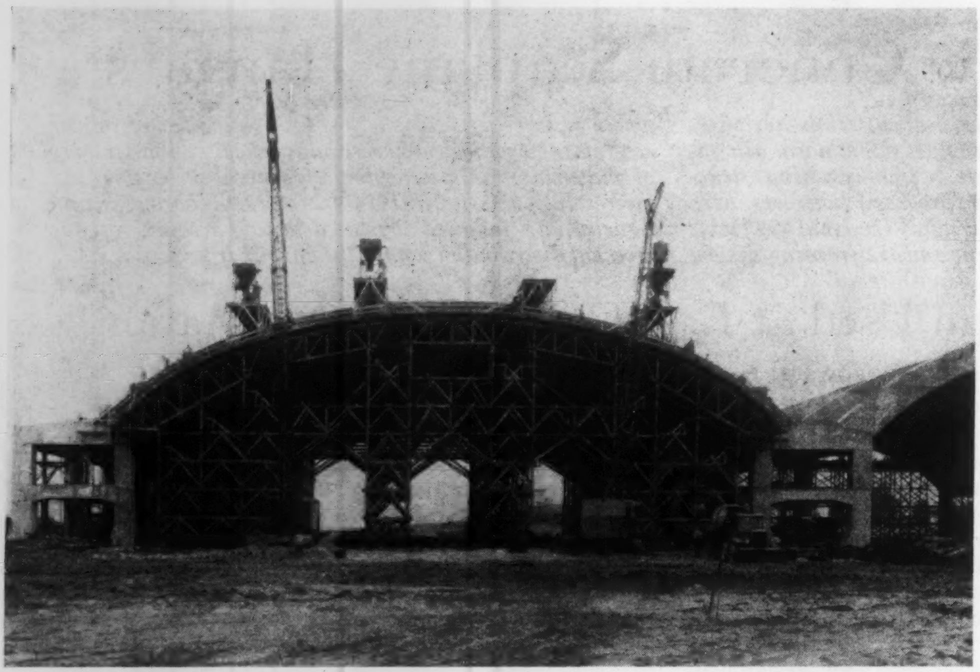
The preparation of the ite for each of the six hangars constructed varied, and the following detailed description applies specifically to the first hangar contructed. The site was fixed in an area requirng heavy cut. The overburden ranged from 7 to 0 ft. The ground was leveled to an elevation just below final floor level and test borings were made. Before the footings were poured, an additional cut was made to the approximate level of the bottom of the footings.

The soil is described as coarse, brownish-gray sand and gravel with a resistance against penetration (from test borings) of 60 to 70 blows per ft. The high and low elevation of this strata varied by 10 ft. The Bureau of Yards and Docks analyzed the borings and undisturbed samples and made recommendations as to allowable soil pressure, expected settlements, and frictional resistance of the soil. For mean conditions, with uniform pressure over the footing area, 4,000 lb per sq ft was taken as the allowable soil pressure. A maximum of 6,000 lb per sq ft was permitted for eccentric loadings of short duration. A frictional resistance equal to one-third the vertical load was used to determine the lateral resistance against horizontal thrust. For each arch two galvanized bridge strands $1^9/_{16}$ in. in diameter, with an ultimate capacity of 300,000 lb per strand, were used to take 200,000 lb of the horizontal thrust at a working load of 100,000 lb per strand. Where maximum loading conditions gave a horizontal thrust exceeding that provided for by frictional resistance and bridge strands, consideration was given to the passive resistance of the soil.

Adoption of spread footings shortened the construction period and reduced the cost of foundations. The safety of putting spread footings on a rather dense stratum overlying a stratum of soft silt was studied by the Bureau of Yards and Docks. Tests of undisturbed samples indicated that the silt could carry the reduced unit loads computed on a bearing area at the silt elevation formed



Double Seaplane Hangar of Reinforced Concrete



ROLLING SCAFFOLDING USED FOR ARCH-RIB AND ROOF-SLAB FORMS

by the intersection of a horizontal plane with 45-deg planes projecting downwards from the edges of the bottom of the footings.

A further study was made to determine the practicability of individual spread footings. A mean eccentricity was assumed and the footing so proportioned that the center of gravity of the footing mass coincided with the thrust line to give a uniform soil pressure over the footing area. At the same time the shape was modified to keep the soil pressure below the maximum under extreme positions of the thrust line. These conditions resulted in an overlapping of footings. Combined footings were necessary.

Center leanto footings were made rectangular because final loading conditions give zero eccentricity under uniform loads. The construction procedure permitted pouring one roof section at a time; hence, the footing was provided with enough area to satisfy construction loads. The footings within and at the end of the building are similar in shape and modified only by the difference in intensity of load.

DESIGN OF ARCHES

The architectural plan of this building was fixed by Navy standards for wood hangars. While the spans were essentially fixed, a certain amount of flexibility was possible in the shape of the hangar roof. The final curvature realized a dual purpose since it fulfilled structural requirements and permitted the reuse of wood centering from a former 200-ft hangar.

Four methods of framing were studied in considering the problem of bringing the arch reactions to the footings: (1) single columns inclined from the footings to the springing line of the arches; (2) single columns vertical from the footings to the springing line of the arches; (3) inverted V-frames combining inclined columns and vertical columns; (4) two-story bents.

The first method utilized the economy of arch action over the full span. It was found, however, that the saving in arch steel for this short span was not enough to offset the expense of forming high inclined columns. Furthermore, the cost of using tension ties would be raised because the tie length would be greater than that fo either of the other solutions.

As for the second method, i simplified column forming and utilized the leanto areas more economically, but required more reinforcing steel and concrete. Where method No. 1 would provide a structural unit that was essentially an arch method No. 2 would provide a structural unit that was essentially a frame.

Consideration was given to the third method in an effort to reduce the construction expense of a single inclined column requiring extensive shoring. Investigation showed, however, that the arches were not benefited by the addition of a vertical post. Loadings could occur that would produce uplift on this post. The combined column areas and the angle at which the inclined column penetrated the leanto resulted in a loss of space and complicated the full utilization of leanto facilities.

The fourth and adopted method offered the advantages of both of the first two methods. The two-story bents are massive and tend to fix the arch at the springing line. They are easily formed. Using travelers for the second-story slab and the roof slab, the contractor was able to pour the leantos well in advance of the main hangars. The inclined column design has been used on larger-span hangars, and the single vertical column design has been used on hangars not requiring large leanto areas. In all these cases the jobs were awarded after careful consideration had been given to conventional designs in steel, wood, and other types of concrete framing.

Finally, the various schemes were compared from the standpoint of safety. The monolithic nature of the adopted design and the rather unfavorable footing conditions required comprehensive studies of the effects of footing rotations, settlements, and horizontal displacements. Conservative provisions were made for these effects, thus gaining in overall safety for the condition of dead load plus live load when compared with the other three methods.

Thought was given to the position of the shell at its connection with the arch. Construction speed and economy were greatest with the shell at the arch intrados, because of the simple and unobstructed movement of the rolling centering.

The design of the final structural unit, two arches resting on three two-story bents, is a problem 24 times statically indeterminate. Because the exterior bents are identical in shape, the number of redundants is reduced to 18. These are further reduced to 12 when a symmetrical loading system is involved, taking into account dead load and temperature change or shrinkage. The construction procedure introduced a design involving 15 redundants, a single arch supported on two two-story bents.

An important consideration in the design of a thin curved roof slab is its shape. Structural considerations of load, span, buckling stability, and edge conditions resulted in a curve of three radii. The center five-eighths of the span has a radius of 165 ft, and the remainder has a 65-ft radius. A small section of transition radius equal to 100 ft is used.

Since the leantos occupy nearly 25% of the total covered area of the building, it was important that work should progress on them without waiting for the main hangar roofs. The contractor favored the use of travelers for the second-floor slab as well as for the roof slab. Although the flat leanto roof would eventually be made monolithic with the curved shell, it was necessary to make this roof strong enough to carry itself and the construction loads independently of the shell. The term "self-supporting" will hereafter be applied to this structural requirement. The alternate solution would require that both the flat roof slab and the curved shell be supported by centering until such time as the centering could be simultaneously lowered under both. This would produce an interaction between the two slabs under dead load and all subsequent loads. The term "non-self-supporting" will hereafter be applied to this alternate solution. The stiffening rib between the arches of each unit was not required in the region of the shell formed by the 65-ft radius. The small radius and short span were adequate insurance against buckling. By cutting the rib short of the shell springing line, a simplification of the construction work could be achieved.

Two-Story Bents Supporting Roof Arch House Shops and Offices

A complete investigation was made involving three separate designs: (1) leanto roof not self-supporting with stiffening rib continuous, (2) leanto roof self-supporting with stiffening rib stopping at the upper end of the 65-ft radius, (3) leanto roof self-supporting with stiffening rib continuous. It was found that approximately 6 tons of steel per hangar could be saved by using either design No. 2 or No. 3. Design No. 3 was finally adopted with the modification that the rib was continued some distance below the upper end of the 65-ft radius. This had the effect of reducing some of the secondary moments in the crown area of the shell. The stiffening rib at the end of the shell cantilever had to be made continuous, since it would be dangerous practice to leave the edge of such a slab unthickened. It was also necessary in order to make a satisfactory job of keeping this expansion joint watertight.

For the case of uniform symmetrical loading, the shell is 8 times statically indeterminate. The design was made in successive steps. First, continuity was established between the flat leanto roof and the shell. Then the shell was cut at the change of radius and another set of redundants was computed in order to restore equilibrium and continuity at the section. These redundants introduce dampened waves of stress, which unbalance sections already balanced. This is similar to the system of solving problems by moment distribution. The second set of redundants introduced by the reflected stresses were less than 10% of the magnitude of the primary redundants.

Results of the computations for the adopted design are given in Fig. 1. Diagrams (*a*), (*b*), (*c*), and (*d*) show the final ring stresses (T_2), shearing stresses along the arch (s), longitudinal stresses along the arch (T_1), and transverse moments (M_2), respectively. A coordinate system was then laid out covering a given shell unit, and the principal stresses were computed in accordance with the laws of simple mechanics. Next, the directions of the principal stresses were determined in order to lay out the primary shell reinforcing. These trajectories are plotted in diagram (*e*), where solid lines denote tension and dashed lines, compression. Diagram (*f*) shows a typical shell element, with the direction of the normal stresses and moments.

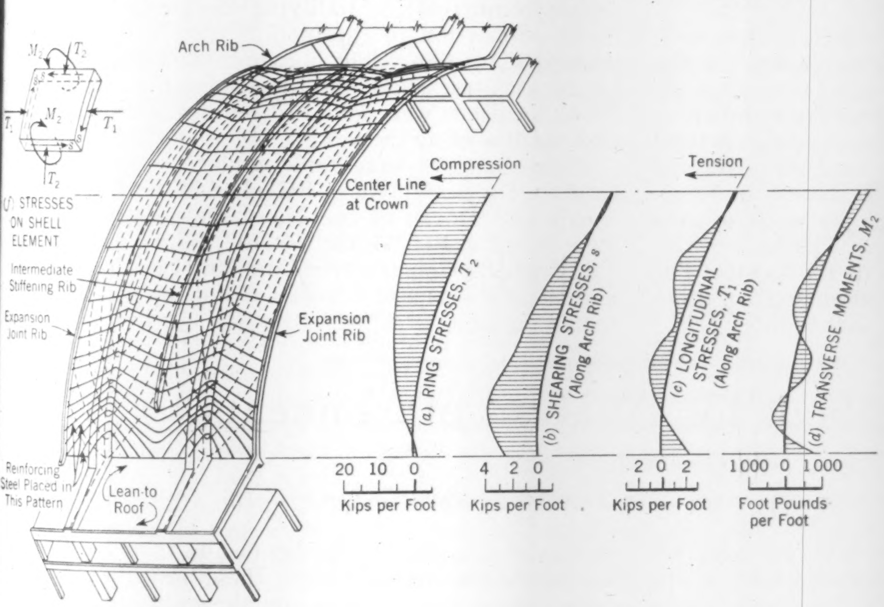

M_2
T_2
s
T_1
M_2
T_1
s
T_2
(f) STRESSES ON SHELL ELEMENT
Arch Rib
Intermediate Stiffening Rib
Expansion Joint Rib
Reinforcing Steel Placed in This Pattern
Lean-to Roof
Expansion Joint Rib
(e) TRAJECTORIES OF PRINCIPAL STRESSES
Compression
Center Line at Crown
Tension
(a) RING STRESSES, T_2
(b) SHEARING STRESSES, s (Along Arch Rib)
(c) LONGITUDINAL STRESSES, T_1 (Along Arch Rib)
(d) TRANSVERSE MOMENTS, M_2
20 10 0
Kips per Foot
4 2 0
Kips per Foot
2 0 2
Kips per Foot
1000 0 1000
Foot Pounds per Foot

Fig. 1. Section of Hangar Showing Principal Stresses

The thin curved slab used in this design (with a cross-sectional curve made up of circular arcs) carries itself and superimposed loads by direct stress action (tension and compression). The primary shell analysis is used in the design of the diagonal steel. Secondary transverse and longitudinal moments are then investigated. Temperature differences between the arch and this shell produce a longitudinal bending in the shell and account for the major part of the steel provided for cross bending. In this instance, as well as for stress differences due to dead and live load, the economy of providing for these moments is not dependent on the position of the shell.

A minimum factor of safety against buckling equal to 6.5 exists under full load at the crown between the arch and the stiffening rib. This value increases rapidly towards the springing line of the shell. In order to decenter the shell safely, the strength and modulus of elasticity are determined by cylinder and beam tests before the centering is lowered.

At each end of the hangar there is a door girder consisting of a horizontal concrete slab supported by vertical struts made monolithic with the end arches. The door guides are mounted on the under side of the slab. The wind reactions are carried by the struts into the arches. Since the arch is thin, this twist cannot be carried by the arch itself down to the bents. The shell is thickened to approximately 10 in. along the face of the arch and designed to carry this moment.

The complete superstructure of one building, covering an area of 103,870 sq ft, required 534 tons of reinforcing steel and 6,100 cu yd of concrete. This is equivalent to 10.27 lb of reinforcing steel per sq ft of covered area, or 8.39 lb of reinforcing steel per sq ft of usable floor space. The latter figure includes 23,389 sq ft of second floor area in the leantos. Putting the concrete on a comparable basis gives 19.0 in. of concrete per sq ft of covered area, and only 15.5 in. of concrete per sq ft of usable floor space. The complete substructure, including footings, grade beams, and foundations for the door tracks, required an additional 130 tons of reinforcing steel and approximately 2,560 cu yd of concrete. Adding the items for the substructure and superstructure gives 664 tons of reinforcing steel and 8,660 cu yd of concrete. These figures do not include the concrete ground-floor slab.

Enough falsework and forms were erected so that work could proceed on one complete transverse unit of the building 71 ft in length. This included six sets of travelers for the leantos (three for the second floor and three for the roof) and two sets for the hangar proper. A total of 560,000 fbm of lumber is reused four times on one building and then used again for a second building. This amounts to 2.70 fbm of lumber per sq ft of area covered during the use of the lumber. The bulk of this lumber is salvageable on completion of the work. The contractor had shipped to the site lumber used on other shell-roof projects amounting to 425,000 fbm.

A final comparison can be made between this type o construction and the more conventional designs in structural steel and wood. At this same site a steel buildin with wood decking required 24.2 lb of steel and 2.93 fbm of lumber per sq ft of covered area. A wood building required 3.5 lb of steel, 22.0 fbm of lumber, and 7.75 in. o concrete per sq ft of covered area. The rather hig quantity of concrete is due to the need for concrete pylon 60 ft high at each end of the leantos supporting the wooden door girders. The quantities given are only for the superstructure. These quantities are not intended to convey the impression that such designs cannot be made with less material, or with a different distribution of materials. They do represent actual requirements on a basis of similarity in span, loading, and door openings at the same site.

A subject of this nature can be a matter for endless argument. Job conditions vary the progress rate even at the same site. The project required speed of erection The time figures given here are for a typical double hangar, one of six, constructed simultaneously with the others. The footings were started April 5, 1943, and completed April 30. The leantos were started May and finished June 23. The shell roofs of the hangars were started May 29 and finished June 29, 1943. Eleven weeks after the first footings were poured the last section o roof was poured. Speed in this type of construction depends on the cyclic reuse of forms, and in this way the principles of mass production are applied to construction The cost of these hangars and the length of time necessary for their construction indicate the competitiveness of monolithic concrete.

A structure as highly indeterminate as this naturally leads one to question the exactness of the designer's analysis. Opportunity and time have not permitted extensive measurements, but deflections of the crown o the arch have been recorded at the time of decentering The calculated deflection there, under the final dead-load condition, based on a modulus of elasticity o 3,000,000 lb per sq in., is 0.628 in. On April 10, 1943, after decentering the second hangar roof, the Resident Engineer reported a deflection of 0.985 in. in the crown for a modulus of elasticity of 2,000,000 lb per sq in. (based on a simple beam test). Modifying the computed value in direct proportion to the moduli gives a theoretical deflection of 1.076 in. The horizontal movement of several footings was measured and the displacements from established locations were found to be of the order of a few thousandths of an inch.

These hangars were build under the supervision o Captain Ira P. Griffen, (CEC), U.S.N. for the Bureau o Yards and Docks of the Navy Department, of which Rear Admiral Ben Moreell, (CEC), U.S.N., is the Chief The Corbetta Construction Company was the contractor and the Roberts and Schaefer Company was the architect-engineer.

Two Methods for Analysis of Moments

By OTTO GRUENWALD

STRUCTURAL ENGINEER, ROBERTS AND SCHAEFER COMPANY, CHICAGO, ILL.

CONSIDERABLE simplification in the design of the highly indeterminate structure planned for the Navy hangars was secured. After cutting the arches free of the bents, each structural unit was investigated independently under the action of unit horizontal forces and unit moments. The moment-distribution method was applied to the bents, and the slope-deflection method to the arches. Once these basic units had been investigated, the degree of complexity necessary to calculate the horizontal forces and moments required for geometrical continuity under any system of loading never involved solving more than three simultaneous equations.

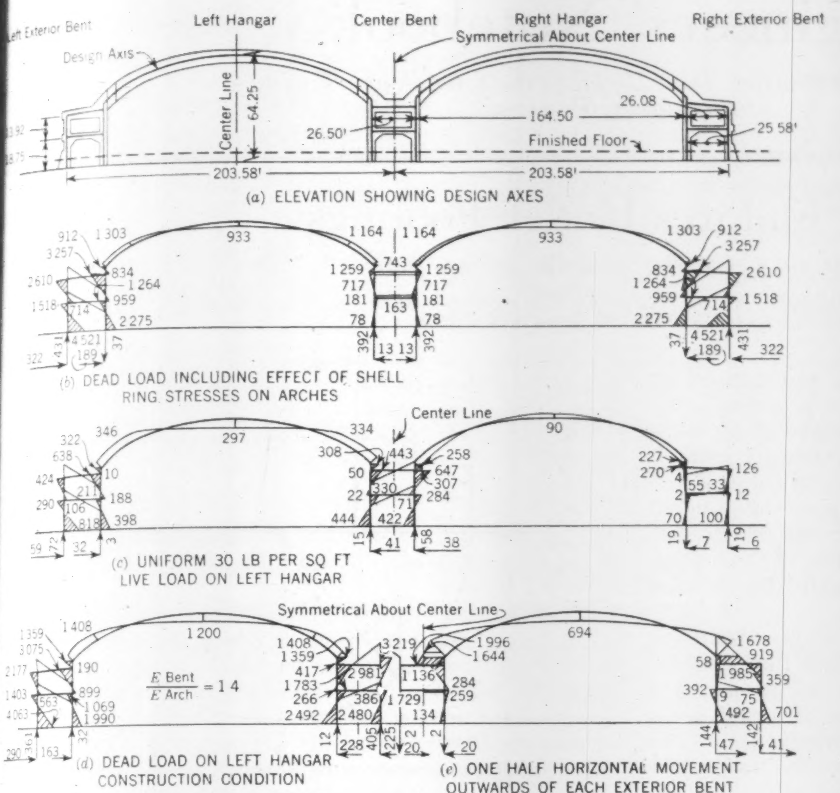

Left Exterior Bent
Left Hangar
Center Bent
Right Hangar
Symmetrical About Center Line
Right Exterior Bent
Design Axis
Center Line
64.25
164.50
26.08
26.50'
25.58'
Finished Floor
203.58'
203.58'
(a) ELEVATION SHOWING DESIGN AXES
(b) DEAD LOAD INCLUDING EFFECT OF SHELL RING STRESSES ON ARCHES
(c) UNIFORM 30 LB PER SQ FT LIVE LOAD ON LEFT HANGAR
Center Line
Symmetrical About Center Line
E Bent / E Arch = 1.4
(d) DEAD LOAD ON LEFT HANGAR CONSTRUCTION CONDITION
(e) ONE HALF HORIZONTAL MOVEMENT OUTWARDS OF EACH EXTERIOR BENT
(f) 1 IN. SETTLEMENT OF BOTH COLUMNS LEFT EXTERIOR BENT
Note: Moments in Foot Kips
(g) 1 IN. SETTLEMENT OF OUTSIDE COLUMN OF LEFT EXTERIOR BENT
Symmetrical About Center Line
(h) 40° F UNIFORM TEMPERATURE INCREASE IN ROOFS OF HANGARS AND BENTS
(k) 40° F UNIFORM TEMPERATURE INCREASE IN SECOND FLOORS OF ALL BENTS

FIG. 2. MOMENT DIAGRAMS AND FOOTING REACTIONS—MOMENTS IN FT KIPS

Moments of inertia were computed for the unreinforced section. The moment of inertia of any beam element beyond the face of a column was taken as infinite; similarly, the moment of inertia of any column element beyond the face of a beam was taken as infinite. The effective width of the shell was considered in determining the elastic properties of the arch although its influence is not great. Rotations and deflections were calculated from sums replacing integrals and with closely spaced elements.

Investigation proved that the magnitude of the arch and leanto moments induced by the eccentricity of the shearing forces from the shell is not great when compared to the moments produced by vertical load (Fig. 2). The primary influence of the ring stresses in the shell, regardless of the position of the latter, is the addition of a tensile force acting normal to the arch cross section. The increase in structural efficiency produced by a decrease in the moments caused by the eccentricity of the normal force, is of secondary importance. For this reason the economical location of the shell and arch connection was governed primarily by simplicity in construction procedure.

Since the bents were poured in advance of the arches, the concrete in the bents had a higher modulus of elasticity than that in the arches at the time when the roof centering was lowered. Computation for the dead-load construction condition therefore assumed a ratio of E (bent) to E (arch) $= 1.4$. All other computations were based on a uniform modulus of elasticity.

In the early discussion of various types of columns, it was asserted that the two-story bents were stiff enough to make the arch economical. The greatest economy of reinforcing steel requires that the thrust line of the arch follow the axis closely. The dead-load horizontal thrust can be compared with the horizontal thrust of a parabolic arch under uniform load. Take

wl = total vertical reaction
l = projected length of arch, springing line to springing line
h = rise of arch axis

For the parabolic arch the horizontal thrust is $wl^2/8h$. For the arch attached to the two-story bents, this thrust is reduced to $wl^2/10.4h$. It is of interest to know that the thrust becomes $wl^2/8.03h$ for the condition of full fixity at the springing line.

The crown moment would be reduced to 255 ft-kips, and the moment at the springing line would become 1,308 ft-kips.

The shell loads were included in two steps—the first, as a uniform vertical load per foot of arch; the second, as a tangential force acting at the intersection of the shell and arch face equal in magnitude to the ring stresses acting in the shell for the given condition of loading.

If one or both columns of a single exterior bent (for example, the left one) settle, the arch in the far span (that is, the right hangar) is not affected appreciably. Furthermore, the magnitude of the moment at the footing is relatively unaffected by settlements of the order of 1 in.

The crown moment for the condition of a 40-deg uniform temperature increase in the roofs of hangars and bents added to the final dead-load moment becomes very nearly zero. A 40-deg temperature increase in the second floors of the bents is shown to have little effect on the arch, and the bent moments are very similar to those associated with such action in a simple two-story bent.

Engineers' Notebook

Suggestions and Practical Data Useful in the Solution of a Variety of Engineering Problems

Shear Failures Under Footings

By Paul Andersen, Assoc. M. Am. Soc. C.E.

Major, Civil Affairs Division, Allied Military Government, Supreme Headquarters, Allied Expeditionary Forces in England

While settlements under a footing are caused by consolidation of the supporting soil, actual rupture of the ground is due to a shear failure along some curved surface. A typical failure line generally extends from one edge of the footing down through the soil and then up to some point of the top surface, the tangent generally making an angle of 45 deg with the horizontal. As will be shown, considerable simplification of the mathematical analysis can be obtained by substituting circular arcs for the curved failure line.

In Fig. 1 is shown a footing of width B and transmitting a load P. The shearing resistance of each unit element of the half circle, representing a possible failure line, is $N \tan \phi$, N being the normal pressure intensity and ϕ the angle of internal friction. If only the weight, w, of the overlying soil is considered, it is seen that the total vertical force acting on an infinitesimal element will be

$$r \sin \theta \times r d\theta \sin \theta \times w = wr^2 \sin^2 \theta d\theta \quad \ldots (1)$$

and the normal component on this element will be

$$wr^2 \sin^3 \theta d\theta \quad \ldots\ldots\ldots\ldots\ldots (2)$$

Taking moments about the center of the semicircle gives

$$P\left(r - \frac{B}{2}\right) = \int_0^\pi wr^2 \sin^3 \theta \tan \phi d\theta \times r \quad \ldots (3)$$

Integrating and solving for B,

$$B = 2r - \frac{8wr^3 \tan \phi}{3P} \quad \ldots\ldots\ldots\ldots (4)$$

To each value of r will correspond a value of footing width, B, over which the foundation pressure must be distributed in order to avoid failure along the corresponding semicircle. The most dangerous failure line is the one which gives the largest value of B. If B is large enough to prevent failure along this line, it will automatically satisfy all other possible failure lines.

$$\frac{dB}{dr} = 2 - \frac{8wr^2 \tan \phi}{P} = 0; \; r = \sqrt{\frac{P}{4w \tan \phi}} \quad \ldots (5)$$

Substituting this expression in Eq. 4 gives

$$B = \frac{2}{3}\sqrt{\frac{P}{w \tan \phi}} = \frac{3}{4}r \quad \ldots (6)$$

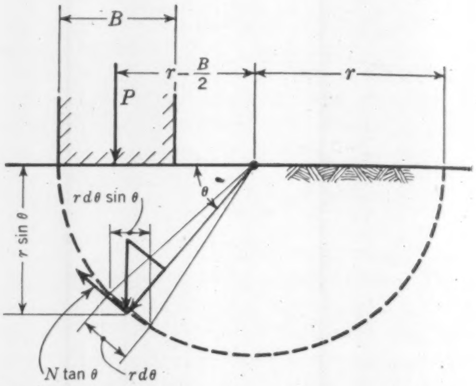


Fig. 1. Representation of Forces Causing Shear Failure of Soil Under a Footing

This expression is considerably on the safe side, as no account was taken of the increased shearing resistance due to the footing load, P, on portions of the plane of rupture. As Eq. 8 shows, the most dangerous radius is slightly smaller than the footing width, B. It can therefore be assumed without appreciable error that the load P is uniformly distributed over one quadrant of the semicircle. Taking moments about the center of the semicircle gives

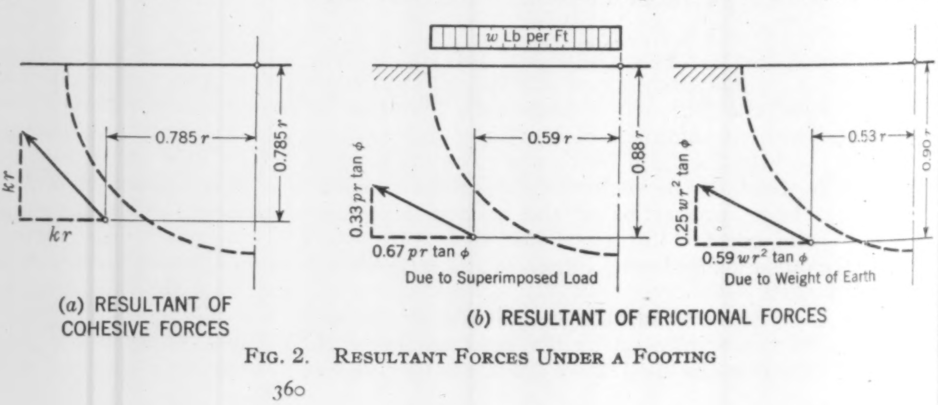
$$P\left(r - \frac{B}{2}\right) = \int_0^{\pi/2} \frac{P}{r} \sin^2 \theta \tan \phi r d\theta r + \int_0^\pi wr^3 \sin^3 \theta \tan \phi d\theta \quad \ldots (7)$$

Proceeding as before, the minimum width B, of the footing is found to be

$$B = \frac{2}{3}\left(2 - \frac{\pi}{2} \tan \phi\right)\sqrt{\frac{2 - \frac{\pi}{2} \tan \phi}{8w \tan \phi} P} \quad \ldots\ldots (8)$$

NUMERICAL EXAMPLE

It is desired to find the required width for a bridge pier 40 ft long, transmitting a load of 1,000 tons. The soil weighs 100 lb per cu ft and has an angle of internal friction equal to 30°. The load, P, per lin ft, will equal 50 kips.



(a) RESULTANT OF COHESIVE FORCES

(b) RESULTANT OF FRICTIONAL FORCES

Fig. 2. Resultant Forces Under a Footing

Substituting in Eq. 8,

$$B = \frac{2}{3}\sqrt{\frac{\left(2 - \frac{\pi}{2}\,0.577\right)^3}{800 \times 0.577}\,50{,}000} = 7.91 \text{ ft}$$

If a factor of safety of 2 is assumed, the footing width should be made 15.8 ft.

Soil cohesion, which does not depend on the value of the normal stress, can be taken into account by subtracting another term from Eqs. 4 and 8. From Fig. 2 it is seen that this term should be

$$kr \times 0.785 \times 4 \times \frac{2}{P} = 6.28\,\frac{kr^2}{P} \quad \ldots\ldots(9)$$

In case of a footing resting at a subsurface level, a possible failure line can be approximated by two quarter-circles whose radii have a difference equal to the depth of the footing below the surface. If only friction is present along this failure line, resistance to rupture can be expressed by the resultants in Fig. 2(*b*). In case of cohesion, the information given in Fig. 2(*a*) should also be taken into account.

Restricted Conditions in Direct Design of Indeterminate Trusses

By A. G. STRANDHAGEN, JUN. AM. SOC. C.E.

ASSISTANT PROFESSOR OF MECHANICS, CARNEGIE INSTITUTE OF TECHNOLOGY, PITTSBURGH, PA.

In the application of the method of direct design to indeterminate trusses, several inconsistencies may occur. By a study of the necessary and sufficient conditions, these inconsistencies are rectified. Consequently, other interesting conclusions and simplifications are found. To demonstrate the conditions, it is necessary to recall briefly the principles underlying the method of direct design.

Considering a pin-jointed framework of n members, where the loads and reactions occur at the joints only, the strain energy U of the framework is

$$U = \sum_{i=1}^{n} \frac{F_i^2 l_i}{2A_iE_i},\ (i = 1, 2, \ldots n) \quad \ldots\ldots(1)$$

where F_i is the force in the bar i; l_i, A_i, and E_i are the length, area, and modulus of elasticity, respectively, of bar i.

Extra bars inserted in the structure will be redundant and the forces in the resulting structure can no longer be found by a stress diagram or by simple resolution of forces. Let the unknown forces in these redundant bars be denoted by $X_1, X_2, \ldots$ Assuming the principle of superposition, the forces in any bar i will in general be of the form

$$F_i = a_iP_1 + b_iP_2 + \ldots + u_iX_1 + v_iX_2 + \ldots \quad \ldots\ldots(2)$$

where $a_i, b_i, \ldots, u_i, v_i, \ldots$ are the numerical coefficients depending only upon the geometry of the framework, which can be calculated for the structure by statics. Thus a_i is the force in bar i due to a unit load acting at the same point and in the same direction as P_1, with $P_2, \ldots$ and $X_1, X_2, \ldots$ removed. A similar procedure is carried out for b_i, etc. The coefficients u_i are the forces in the bars due to two oppositely directed unit loads acting in the direction of the redundant member X_1, where $P_1, P_2, \ldots$ and $X_2, \ldots$ are removed. A similar procedure is carried out for $v_i, \ldots$.

By the principle of least work,

$$\frac{\partial U}{\partial X_1} = 0,\quad \frac{\partial U}{\partial X_2} = 0, \quad \ldots\ldots(3)$$

Applying Eq. 3 to Eq. 1,

$$\left.\begin{aligned} \frac{\partial U}{\partial X_1} &= \sum_{i=1}^{n} \frac{F_il_i}{A_iE_i}\frac{\partial F_i}{\partial X_1} = 0 \\ \frac{\partial U}{\partial X_2} &= \sum_{i=1}^{n} \frac{F_il_i}{A_iE_i}\frac{\partial F_i}{\partial X_2} = 0 \end{aligned}\right\} \quad \ldots\ldots(4)$$

Calculating $\partial F_i/\partial X_1$, $\partial F_i/\partial X_2$, $\ldots$, from Eq. 2, and substituting in Eq. 4 the following is obtained,

$$\left.\begin{aligned} \sum_{i=1}^{n} \frac{(a_iP_1 + b_iP_2 + \ldots + u_iX_1 + v_iX_2 + \ldots)u_il_i}{A_iE_i} &= 0 \\ \sum_{i=1}^{n} \frac{(a_iP_1 + b_iP_2 + \ldots + u_iX_1 + v_iX_2 + \ldots)v_il_i}{A_iE_i} &= 0 \end{aligned}\right\} \quad (5)$$

It is to be noticed that the cross-sectional area of all bars is required as well as the modulus of elasticity; the latter restriction can be removed if $E_i = E$ $(i = 1, 2, \ldots n)$, that is, the material of which the structure is made is the same throughout. By noting that

$$s_i = (a_iP_1 + b_iP_2 + \ldots + u_iX_1 + v_iX_2 + \ldots)/A_i \quad \ldots(6)$$

Eqs. 5 take the following form:

$$\left.\begin{aligned} \sum_{i=1}^{n} s_iu_il_i &= 0 \\ \sum_{i=1}^{n} s_iv_il_i &= 0 \end{aligned}\right\} \quad \ldots\ldots(7)$$

A series of equations corresponding to Eqs. 7 can now be written, the number of which will be determined from the number of redundant members.

In the method of direct design Eqs. 7 are used, whereas the older method of approach uses Eq. 5. Solution of the simultaneous equations represented by Eqs. 5 leads to the evaluation of $X_1, X_2, \ldots$, from which the corresponding stresses in the truss are found. Usually the stresses are of a wide range, notwithstanding the fact that they are often above the design stress, and hence the assumed areas are changed and the process repeated. This leads to a process of trial and error in addition to laborious calculations. A more desirable and direct method is to use Eqs. 7 by stipulating the stresses that each member must withstand, thus the method of direct design. However the satisfying of Eqs. 7 does not necessarily indicate that the assumed stresses are possible.

It is the purpose now to show that Eqs. 7 are not satisfied until the stresses computed are the assumed stresses in magnitude and sense. Application of the necessary and sufficient conditions is shown in the following example.

The rectangular frame shown in Fig. 1 (*a*) is subject to loads P and Q. Removing the external loads and replacing the redundant member by a unit load, the u_i coefficients are obtained. Eqs. 7 become with u_i and l_i from the preceding diagrams

$$-s_1 l \sin^2\theta - s_2 l \cos^2\theta - s_3 l \sin^2\theta - s_4 l \cos^2\theta + s_5 l + s_6 l = 0 \ldots\ldots\ldots (8)$$

For economical design, it is desirable to have the members carry the same magnitude of stress, thus assuming

$$\left.\begin{array}{l} s_2 = s_3 = s_5 = +s \text{ (tensile stress)} \\ s_1 = s_4 = s_6 = -s \text{ (compressive stress)} \end{array}\right\} \ldots (9)$$

Then Eq. 8 becomes

$$(s - s)l \sin^2\theta + (s - s)l \cos^2\theta + (s - s)l = 0 \ldots\ldots (10)$$

Equation 10 indicates that regardless of the magnitude of the stresses assumed, Eqs. 7 are satisfied for all values of θ, providing the bars are stressed in equal magnitude in compression and tension, and that certain bars be stressed as indicated by Eqs. 9. The calculations for a_i and b_i are shown in Figs. 1 (*c*) and (*d*).

From Fig. 1 (*a*) and (*b*) the force in any bar of the truss can be obtained as a function of X, the unknown redundant force. Similarly by Eq. 6 the stress is likewise obtained. In order to insure that the assumed conditions of Eqs. 9 are adhered to with proper sign conventions (sufficient conditions), it is necessary to analyze each bar for the range of X. Since $\sin\theta > 0$ and $\cos\theta > 0$ for $(0 < \theta < \pi/2)$ and $A_i > 0$ $(i = 1, 2, \ldots n)$, then for

BAR 1: $-X \sin\theta = -sA_1$, or any value of $X > 0$ satisfies above.

BAR 2: $P - X\cos\theta = +sA_2$, or provided $X/P < \sec\theta$.

BAR 3: $P\tan\theta - X\sin\theta = sA_3$, provided $X/P < \sec\theta$.

BAR 4: $P - Q - X\cos\theta = -sA_4$, provided $X/P > (1 - Q/P)\sec\theta$.

Similarly for Bar 5 and Bar 6 whose restrictions are the same as Bar 1 and Bar 2 respectively.

Where $Q < P$, a graph for this design is shown in Fig. 1 (*e*). The permissible range for X as a function of θ, that is, $X(\theta)$, is shown by the shaded area. Given P, Q, and θ where $Q < P$, X is computed which will insure the satisfying of conditions (9) and (8), as well as Eqs. 7. It thus appears that the selection of the stress conditions assumed in Eqs. 9 was not sufficient unless the force in the redundant member $X(\theta)$ was restricted in range, hence necessary but not sufficient conditions.

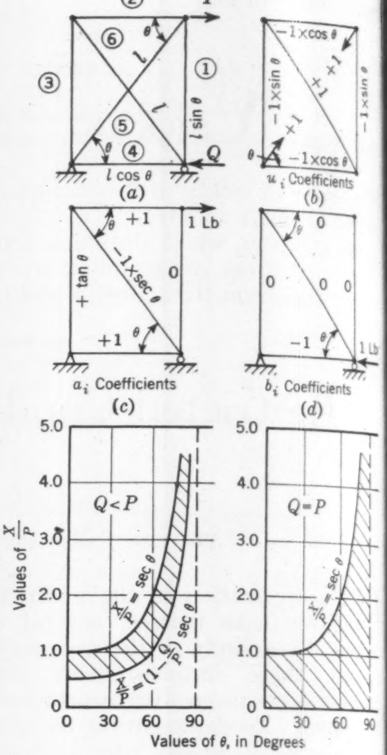


FIG. 1. RECTANGULAR REDUNDANT FRAME WITH SOLUTION FOR FORCE IN ANY MEMBER

If $Q = P$, it can be seen that the range for X is $(0 < X/P < \sec\theta)$ shown by the shaded area in Fig. 1 (*f*).

If $Q = 0$, there are two inconsistencies which cannot be satisfied, that is,

$$X/P < \sec\theta \text{ and } X/P > \sec\theta$$

No value of X will satisfy these conditions. Hence the satisfying of Eq. 8 is necessary but not sufficient. Further study of this specific problem, $Q = 0$, will reveal, by using a parallel to the foregoing, that this is an impossible case either with the conditions of Eq. 9, or with these stresses taken in different order. Hence it is impossible to have the stresses equal in magnitude in all bars.

An analysis of this kind clearly indicates what design is possible or impossible. It also indicates the range in which X may take permissible values, a valuable aid if the designer is confronted with elastic buckling.

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Suggested New Bidding for Building Materials

TO THE EDITOR: A short item of interest in the June issue of CIVIL ENGINEERING (page 274) announced a proposal by the Producers' Council, Inc., for a new bidding practice for building materials to replace the controversial "or equal" clause. This proposal has created widespread interest. It has been generously approved by some and as vigorously opposed by others. Several very pertinent questions have been raised concerning it.

One important question is whether the proposed bidding practice restricts competition. It is primarily to dissipate the latter question that this letter has been prepared. The misunderstanding undoubtedly arises from the fact that one product has to be named by the specifier to govern the basic bid so that all bidders submit strictly comparable basic bids. However, the specifier may name several makes of products in the specifications, and the bidders, in any case, are permitted to offer other products which in their opinion would give equal quality and value (to be furnished at the same price as the product named for the basic bid or at a higher or lower price, as the case may be).

The Council's plan is offered as a substitute for the "or equal" clause in those instances where the architect or engineer customarily uses the "or equal" clause. The new procedure in no way prevents the use of standard specifications in which no product is named. It is proposed for use primarily in private construction, rather than in public construction where present legal limitations might preclude its application. However, if it demonstrates its usefulness in private work, ways and means undoubtedly can be found to adapt it to public bidding.

J. W. Follin, Assoc. M. Am. Soc. C.E.
Managing Director, The
Producers' Council, Inc.

Washington, D.C.

In Protest Against Collective Bargaining

Dear Sir: The writer, as a Life Member of the Society and as a worshipper at its professional shrine for considerably more than half its lifetime, feels obliged to record his protest against the recent actions of the Board of Direction toward "unionizing" the membership.

He believes that these actions are:

1. Contrary to Article One of the Constitution of the Society, and *ultra vires*.
2. Contrary to the spirit and intent of all the previous conduct of the Society and of all the long struggle for its elevation.
3. Un-American (in that they abandon fundamental principle for temporary expediency); they launch a policy of "appeasement," and will prove worse than futile.

The writer further believes that these actions of the Board of Direction and its agents have been bureaucratic in the extreme; that they annihilate all past efforts for the establishment of civil engineering as a learned profession; and that, unless they are promptly repudiated by the Society's membership, they will result in the relegation of civil engineers to the level of skilled technicians.

W. W. Crosby, M. Am. Soc. C.E.

Coronado, Calif.

The Engineer and Postwar Planning

To the Editor. In connection with Colonel Carey's article on postwar engineering projects in the July issue, I am struck by the fact that civil engineers are wasting their suggestions on other civil engineers. Only legislators can effectuate what Colonel Carey suggests. Nevertheless, we civil engineers keep talking to ourselves instead of to our fellow citizens. We continue to limit our circle as though we were Boy Scouts around a camp fire telling each other what we generally know. Let's put our ideas before a public forum, so that something may be done about them.

We admit we know how to originate, plan, design, and specify. Then, why not reveal our professional abilities to groups that can pass laws and ordnances to enable our dreams to become realities? We have had legislators, for example, ask if viaducts are bridges. Whereas if we become more cooperative minded, our legislators will more and more turn to qualified engineers for their engineering terminology and guidance.

The President has forwarded to Congress a report on inter-regional highways. This report is full of technical possibilities, but there is no way of making these possibilities useful to the taxpayers who, in turn, will be needed to tell their congressmen to vote authorization and appropriations for this proposed inter-regional system. Perhaps if we would convert the function of such a highway system into reduced costs of food, clothing, and shelter, we would travel farther and faster.

A parkway is nothing but a highway for those who like attractive roadside development or natural scenic outlooks. When will civil engineers realize that travelways are fundamentally for the traveler and not for the travel vehicle? Though we wouldn't care to ride on freight trains, we continue to ride behind freight trucks exposed to the constant hazard of mixed traffic. Safety in travel is an "American First." Yet in each of the prewar years more people were killed on the highways of America than were killed in England last year by German bombs.

If metropolitan planning is to become really effective, civil engineers will have to divert the mind of the average citizen away from man-made city limits. The boastful local pride of the average person is generally unwarranted, because he has usually contributed nothing toward the development of his urban area. "Metregional" planning is one of the great engineering opportunities of tomorrow.

While attending the International Housing and Planning Congress in Stockholm in 1939, I was amazed by the evidence that re-urbanization of large cities everywhere was considered essential. Since then some of these antiquated urban areas have been bombed out of existence, and rebuilding them will require civil engineering of the broadest type.

Another field for postwar planning is the coordination of terminals. To expedite the movement of passengers and freight and to decrease operating expenses, terminals for all the different ways of transit must be coordinated and their facilities unified. The great savings that would result must become realities in the postwar world.

Let us suggest to church wardens that houses of worship are important enough to be surrounded with ample well-landscaped grounds. There is plenty of land in America to do it. Let us tell the doctors that we are ready to build a hospital in every county as a postwar memorial to their local war heroes.

We are spending an average of $100 per capita per month for global war. What shall we budget for postwar projects? That, civil engineers must help decide. The foregoing suggestions are merely indications of what our postwar plans can be. They will be small or great, depending upon our ability to enlist legislators in the cause of projects suitable to a great new technical era.

A. P. Greensfelder, M. Am. Soc. C.E.
Chairman, Fruin-Colnon
Contracting Company

St. Louis, Mo.

Forum on Professional Relations

Conducted Column of Hypothetical Questions with Answers by Dr. Mead

In the current issue Dr. Mead gives his answer to Question No. 23, which was stated in the June issue of "Civil Engineering." The question reads as follows: "A young engineer, 'A,' is employed by a certain firm and is—by verbal agreement—to remain with the company for two years. At the end of the first year, he is approached by another firm which desires to employ him. 'A' does not mention his agreement with the first firm but suggests a salary which he considers so exhorbitant that it would not be considered. The new firm, however, is satisfied with the salary mentioned and agrees to it if the young engineer will immediately undertake the work. What should the young engineer do?"

Any agreement should be carefully considered before it is made and, once made, it should be duly carried out. Dependability is the primary requirement for any successful engineer, and if he is willing to break his word for an increased salary he cannot be regarded as a dependable man. In fact, it may be said in general that he has laid the foundation for professional failure.

The engineer, in the case in point, should have stated at once that he had agreed to stay with his employer for another year and was therefore unavailable until the end of that period. His silence on the agreement and the fact that he named a salary gave the impression that he was free to accept a new position at once. He therefore placed himself in an embarrassing position in that he must either accept the new offer, thus breaking his word with his present employer, or explain the situation to him, which he should have done when the prospective new employer first approached him.

Daniel W. Mead, Past-President and
Hon. M. Am. Soc. C.E.

Madison, Wis.

Question No. 24, as given in the July number, will be answered in the next, or September, issue. Next in the series the following question is announced. Replies may be received until September 5, with answers in the October issue.

Question No. 25: *A municipality hires the services of an engineer on a salary basis of $3,000 per year. Is the engineer justified in doing private work during his spare time and keeping the pay therefor for his own use?*

SOCIETY AFFAIRS

Official and Semi-Official

Classification and Compensation Plan Adopted by Board of Direction

Professional civil engineering positions were classified and a compensation plan established by action of the Board of Direction at its meeting in Cleveland on July 18, 1944. In presenting this classification and compensation plan, the Committee on Salaries also recommended its publication with provision for reprints. The report as adopted follows:

Previous Classification and Compensation Plans

In February 1939 Civil Engineering published, for appraisal and discussion, a classification and compensation plan for Civil Engineering Positions, prepared by the Society's Committee on Salaries, which had been presented to the Board of Direction at its January 1939 meeting. In succeeding weeks 24 Local Sections, with membership totaling approximately 7,400, submitted reports, and in addition written comments were received from numerous individual members of the Society.

At the April 1939 meeting of the Board, a special committee of three members of the Board of Direction was appointed to correlate the comments, suggestions, and criticisms received and to report back to the Board with recommendations. This classification and compensation plan was adopted unanimously July 25, 1939.

At the January 16, 1940, meeting of the Board, the Professional Grades I, II, and III were amended to lower the experience requirements by two years.

On October 10, 1943, a report was submitted to the Board, which, although approved by the Board, was returned to the Committee for clarification and enlargement.

Foreword

Professional civil engineering positions should be classified according to relative importance of duties to be performed and to the responsibilities incident thereto. In this report, therefore, a general specification has been established for each classification and is intended to describe the duties and requirements usually associated with the classification and the qualifications expected of the person properly to perform the work and to assume the responsibilities specified.

These classifications should be used with judgment and discretion as to application or adaptability to meet local situations and sectional divergences, which naturally exist within a field that is national in extent. It is expected that in applying this plan to organizations which do not have employees in all of the nine classifications, due consideration will be given to the appropriate relationship of the various positions which exist in such organizations and the duties and responsibilities of such positions. When so used, the following classification and compensation plan will serve as an adequate guide to establish the basic organization requirements as well as the appropriate minimum compensation.

Note that the grade numbers have each been increased by one; that is, new Grade II corresponds approximately to Grade I of the previous system; new Grade III to old Grade II, etc. The new Grade I is thus the first grade in a complete series for professionally minded engineers.

Classification and Compensation Plan for Professional Civil Engineering Positions

Grade I ($1,980 to $2,460)

Definition and Duties: To include all positions, the duties of which involve civil engineering work of minor professional difficulty and little responsibility but which is of the nature of training for advancement into the higher professional grades. This engineer-in-training grade is designed to apply to those beginners in the profession who are professionally minded and whose education or experience will have fitted them, after a suitable training period, to undertake the duties and responsibilities of the higher professional grades. It is not inclusive of those positions which, in their nature, generally will not lead to advancement into the higher professional grades.

Minimum Requirements: Education and Experience. Graduation with a degree in engineering from an engineering school of recognized standing.

Note: Although a person without complete college education in engineering cannot meet the minimum requirements for Professional Grade I, he may, if professionally minded, acquire during his employment with an engineering organization in work corresponding to this grade the necessary training, experience, and self-improvement to enable him to qualify for entrance to Grade II.

Compensation:

Starting Rate $165 a month—$1,980 a year
Full Rate $205 a month—$2,460 a year

Grade II ($2,460 to $3,000)

Definition and Duties: To include all positions involving the performance of professional civil engineering duties under some measure of supervision, and with the opportunity for the exercise of some independent judgment.

Minimum Requirements: Education and Experience. (*a*) Graduation with a degree in engineering from an engineering school of recognized standing plus one year's experience in progressive professional civil engineering work; or (*b*) graduation from high school plus six years' technical engineering experience supplemented by additional self-improvement carried out through study of technical engineering subjects designed for advancement to the higher professional grades; or (*c*) an equivalent combination of (*a*) and (*b*).

Compensation:

Starting Rate $205 a month—$2,460 a year
Full Rate $250 a month—$3,000 a year

Grade III ($3,000 to $3,600)

Definition and Duties: To include all positions involving the performance of professional civil engineering duties, under general supervision, with limited latitude for independent actions and decisions; to perform engineering work of intermediate professional difficulty and responsibility requiring the exercise of independent judgment.

Minimum Requirements: Education and Experience. (*a*) Graduation with a degree in engineering from an engineering school of recognized standing plus at least two years' successful experience in progressive professional civil engineering work; or (*b*) graduation from high school plus at least seven years' technical engineering experience supplemented by additional self-improvement carried out through study of technical engineering subjects designed for advancement to the higher professional grades; or (*c*) an equivalent combination of (*a*) and (*b*).

Compensation:

Starting Rate $250 a month—$3,000 a year
Full Rate $300 a month—$3,600 a year

Grade IV ($3,600 to $4,200)

Definition and Duties: To include all positions involving the performance of professional civil engineering duties under general supervision, and with considerable latitude for independent unreviewed actions and decisions; to perform engineering work of substantial professional difficulty and responsibility; to exercise independent judgment and, in certain cases, to supervise the work of others.

Minimum Requirements: Education and Experience. (*a*) Graduation with a degree in engineering from an engineering school of recognized standing plus at least four years' successful experience in progressive civil engineering work with demonstrated aptitude and capacity for increased responsibilities, one year of which must have been in responsible charge of work; or (*b*) graduation from high school plus at least nine years' technical engineering experience supplemented by additional self-improvement carried out through study of technical engineering subjects designed for advancement to the higher professional grades; or (*c*) an equivalent combination of (*a*) and (*b*).

Compensation:

Starting Rate $300 a month—$3,600 a year
Full Rate $350 a month—$4,200 a year

Grade V ($4,200 to $5,160)

Definition and Duties: To include all positions involving the performance of professional civil engineering duties under general supervision, and with wide latitude for independent unreviewed actions and decisions; to have responsible charge of a subdivision of an engineering organization; to perform difficult engineering work requiring technical training, previous experience, recognized leadership, and the exercise of independent judgment.

Minimum Requirements: Education and Experience. (*a*) Graduation with a degree in engineering from an engineering school of recognized standing plus at least six years' successful experience in progressive professional civil engineering work with demonstrated aptitude and capacity for increased responsibilities, two years of which must have been in responsible charge of work; or (*b*) graduation from high school plus at least eleven years' technical engineering experience supplemented by additional self-improvement carried out through study of technical engineering subjects designed for advancement to the higher professional grades; or (*c*) an equivalent combination of (*a*) and (*b*).

Compensation:

Starting Rate $350 a month—$4,200 a year
Full Rate $430 a month—$5,160 a year

Grade VI ($5,280 to $6,480)

Definition and Duties: To include all positions involving the performance of professional civil engineering duties under general supervision and with wide latitude for independent unreviewed actions and decisions; to have responsible charge of a division of an engineering organization; to perform difficult professional engineering work requiring technical training and previous experience; to exercise independent judgment; to be possessed of the quality of leadership; to direct a staff on design or construction, or to supervise an investigative survey or direct extensive studies of projects.

Minimum Requirements: Education and Experience. (*a*) Graduation with a degree in engineering from an engineering school of recognized standing plus at least six years' successful experience in progressive professional civil engineering work with demonstrated aptitude and capacity for increased responsibilities, four years of which must have been in responsible charge of work, including two years which have demonstrated marked professional attainments; or, (*b*) graduation from high school plus at least eleven years' technical engineering experience supplemented by additional self-improvement carried out through study of technical engineering subjects designed for advancement to the higher professional grades; or (*c*) an equivalent combination of (*a*) and (*b*).

Compensation:

Starting Rate $440 a month—$5,280 a year
Full Rate $540 a month—$6,480 a year

Grade VII ($6,720 to $7,920)

Definition and Duties: To include all positions, the duties of which involve civil engineering work without general supervision and with wide latitude for independent, unreviewed actions and decisions. The duties, knowledge, and skill required for this grade include those covered by Grade VI. They also cover responsibility for a division in a large engineering organization and include duties requiring extensive successful experience in engineering work with demonstrated aptitude and capacity for increased responsibilities in managerial and executive functions.

Minimum Requirements: Education and Experience. (*a*) Graduation with a degree in engineering from an engineering school of recognized standing plus at least six years' successful experience in progressive professional civil engineering work with demonstrated aptitude and capacity for increased responsibilities, at least four years of which must have been in responsible charge of work, including two years which have demonstrated marked professional attainments; or (*b*) graduation from high school plus at least eleven years' technical engineering experience supplemented by additional self-improvement carried out through study of technical engineering subjects designed for advancement to the higher professional grades; or (*c*) an equivalent combination of (*a*) and (*b*). The experience required for this grade cannot be covered by the time limits specified but depends rather on the extent, scope, and quality of such experience.

Compensation:

Starting Rate $560 a month—$6,720 a year
Full Rate $660 a month—$7,920 a year

Grade VIII ($8,160 to $9,360)

Definition and Duties: To include all positions such as the assistant technical and administrative head of an extensive engineering organization, or the technical and administrative head of a smaller engineering organization. It would include a position such as the Principal Assistant to the Chief Engineer.

Minimum Requirements: Education and Experience. (*a*) Graduation with a degree in engineering from an engineering school of recognized standing plus at least six years' successful experience in progressive professional civil engineering work with demonstrated aptitude and capacity for increased responsibilities, at least four years of which must have been in responsible charge of work, including two years which have demonstrated marked professional attainments; or (*b*) graduation from high school plus at least eleven years' technical engineering experience supplemented by additional self-improvement carried out through study of technical engineering subjects designed for advancement to the higher professional grades; or (*c*) an equivalent combination of (*a*) and (*b*). The experience required for this grade cannot be covered by the time limits specified but depends rather on the extent, scope, and quality of such experience.

Compensation:

Starting Rate $680 a month—$8,160 a year
Full Rate $780 a month—$9,360 a year

Grade IX ($9,600 and up)

Definition and Duties: To include all positions such as the technical and administrative head of an extensive engineering organization such as the Chief Engineer.

Minimum Requirements: Education and Experience. (*a*) Graduation with a degree in engineering from an engineering school of recognized standing plus at least six years' successful experience in progressive professional civil engineering work with demonstrated aptitude and capacity for increased responsibilities, at least four years of which must have been in responsible charge of work including two years which have demonstrated marked professional attainments; or (*b*) graduation from high school plus at least eleven years' technical engineering experience supplemented by additional self-improvement carried out through study of technical engineering subjects designed for advancement to the higher professional grades; or (*c*) an equivalent combination of (*a*) and (*b*). The experience required for this grade cannot be covered by the time limits specified but depends rather on the extent, scope, and quality of such experience. This grade requires the highest order of engineering, executive, and administrative ability and outstanding professional attainments.

Compensation:
$9,600 and up

PROPOSED ANNUAL COMPENSATION FOR THE NINE GRADES

CLASSIFICATION	COMPENSATION	
	Annual	Monthly
Grade I	$1,980–$2,460	$165–$205
Grade II	2,460– 3,000	205– 250
Grade III	3,000– 3,600	250– 300
Grade IV	3,600– 4,200	300– 350
Grade V	4,200– 5,160	350– 430
Grade VI	5,280– 6,480	440– 540
Grade VII	6,720– 7,920	560– 660
Grade VIII	8,160– 9,360	680– 780
Grade IX	9,600–and up	800–and up

HOURS OF WORK

The Fair Labor Standards Act of 1938 makes it a Federal law that time and one-half be paid to all non-exempt employees for all time worked beyond 40 hours in any one week. The law provides certain exemptions from the wage-and-hour provisions, especially persons employed in a bona fide "executive," "administrative," or "professional" capacity. Professional employees are exempt from the provisions of the Fair Labor Standards Act of 1938.

The correct professional attitude in the civil engineering profession, as well as in the other professions, is that an employee, unless the law provides otherwise, abides by the hours established by the particular office in which he works or job on which he is engaged. Similarly the correct professional attitude in the civil engineering profession, as well as in the other professions, is that an employer is under obligation to adjust equitably the hours and compensation of those who work for him.

The Government has issued certain war emergency directives which affect salaries, wages, and hours which must be obeyed. They are too involved to be included in a report of this kind. No attempt has been made in the various grades or the requirements, duties, and compensations therefor to correlate them with these directives.

VACATIONS AND SICK LEAVE

Provisions should be made for appropriate sick leave and annual vacation. The length of time for vacation and sick leave will vary with the various services, offices, and jobs and cannot be set up in this report.

ADDITIONAL ALLOWANCES

In individual cases, depending on circumstances, an allowance should be made for moving expenses when an employee is transferred, and for other such extraordinary expenses pertaining to his employment. Under special conditions such as inaccessibility of location, nature of climate, and other exceptional features of employment, the maximum pay for any and all positions in the compensation plan is not to be limited to the stipulated rates.

COLLEGE OF RECOGNIZED STANDING

A college or university of recognized standing is defined as one which is accredited by a national or regional accrediting association such as the Association of American Universities; or the New England, Middle States, North Central, Southern, or Northwest Association of Secondary and Higher Schools; or one whose professional curriculum has been accredited by the Engineers' Council for Professional Development.

PROFESSIONAL POSITIONS IN EDUCATIONAL INSTITUTIONS

The Committee recognizes that a classification and compensation plan should include faculty positions in engineering departments of educational institutions. In order to accomplish this, however, it is recommended that a factual study be made of classifications and salaries of teaching and research engineers in educational institutions.

RETIREMENT SYSTEM

This report does not include a discussion of a retirement system but the Committee recognizes that a sound and adequate retirement system is very desirable and should be, as far as possible, a part of any employment plan.

EXCEPTIONS

While the proposed compensation plan indicates what may be looked upon as appropriate salaries to be paid, it is to be recognized that the capabilities of, and the responsibilities placed upon, many engineers have no definite limitations. Civil engineers, especially those in permanent employment, are frequently called upon and are readily agreeable to working in conformity with the exigencies of a situation. Their compensations, therefore, should be in terms of annual salaries commensurate with the demands made upon them by such exigencies.

It is requested that the membership at large send to Society Headquarters any information or comments which might be helpful to the Committee.

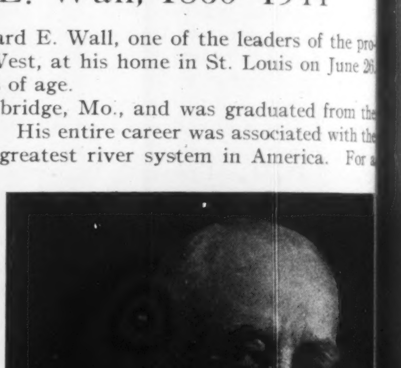
We RECOMMEND the adoption of this Classification and Compensation Plan for Professional Civil Engineering Positions as embodied in this report.

COMMITTEE ON SALARIES
T. E. Stanton, *Chairman*
E. B. Black Charles F. Goodrich
Charles S. Shaughnessy Ernest J. Stocking

Edward E. Wall, 1860–1944

DEATH claimed Edward E. Wall, one of the leaders of the profession in the Middle West, at his home in St. Louis on June 26. He was almost 84 years of age.

He was born in Cambridge, Mo., and was graduated from the University of Missouri. His entire career was associated with the water problems of the greatest river system in America. For a time he was connected with the Mississippi River Commission, but largely his work was in St. Louis, as a servant of the city. This outstanding career of public employment began in earnest in 1892 and ran almost continuously for 50 years. The interruption came in 1926, when a question of engineering standards and propriety caused his temporary retirement. His viewpoint, however, was vindicated when he was reappointed Director of Public Utilities in 1933.


EDWARD E. WALL
Honorary Member of the Society

He brought a high degree of professional integrity to this work, and was thus widely known as representative of the highest type of public servant. His eminent standing was recognized in 1938 when he was elected an Honorary Member of the Society. Public works, and particularly the water works of the City of St. Louis, are his great monument.

Mr. Wall was active in all civic and professional efforts. He served as a Director of the Society from 1918 to 1920, and as Vice-President in 1921 and 1922. Other honors came to him, in keeping with his long and eminent engineering service. He was president of the Engineers Club in St. Louis and of the Institute of Consulting Engineers. In 1935 his alma mater gave him an honorary LL.D. degree.

Discussion of the Principles of Depreciation

In the June issue of **Proceedings** appeared a 16-page tentative report of a special committee appointed by the Board of Direction reating the subject of depreciation as interpreted recently by a committee of the National Association of Railroad and Utilities Commissioners. Because this topic had been selected for the Annual Convention of the Society, which was to have been held in Cleveland, Ohio, prospective discussers were urged to submit comments that could be transmitted directly to the committee.

With the meeting abandoned, however, it becomes feasible to collate written discussions that reach the editor in time, and include them in the September issue of **Proceedings**. Plans are now being formed to include all discussion that can be made ready in time, in a special reprint that will be available for very limited distribution.

Meeting of Board of Direction—Secretary's Abstract, July 17 and 18

The Board of Direction met at the Hotel Cleveland, Cleveland, Ohio, on July 17 and 18, 1944, with the following members present: President Malcolm Pirnie in the Chair; George T. Seabury, Secretary; Past-Presidents Black and Whitman; Vice-Presidents R. E. Dougherty, Hastings, and Thomas; and Directors Bakenhus, Boughton, Breed, Dickinson, N. W. Dougherty, Edwards, Gamble, Goodrich, Hathaway, Lilly, Scobey, Shannon, Tipton, Tolles, and Wilson.

Approval of Minutes

Minutes of the meeting of the Board held on April 24 and 25 at St. Louis, Mo., were approved as written; minutes of the Executive Committee Meeting at St. Louis on April 23, after revision, were also approved as written.

Board and Regional Meetings

Action of the Executive Committee in cancelling the Cleveland Convention, scheduled for July 19 to 21, was approved, with expressions of regret. As regards plans for the fall, the Board decided to hold its meeting at Denver, Colo., during the week of October 8, in conjunction with a meeting of the Colorado Section. It also approved of holding the Local Section Conference on the same occasion.

Tentative plans for spring, summer, and fall meetings in 1945 were discussed.

By-Laws Amended—Article IV

Following regular procedure, the Board approved minor changes in the By-Laws, Article IV, Committees, Section 12 and 12a, by omission of the words "Functional Divisions" and by reference to Article VIII, Section 4, instead of Article VIII, Section 3. The corrected Section will be found in the 1944 Yearbook.

Limitations of Consulting Fees

Acting on more than one request, the Board considered the possibilities of negotiations to increase the statutory fees for consulting services in a number of branches of the federal government, now limited to the amount of $25 per day.

Tribute to Miss Carolina Crook

A minute was adopted in warm appreciation of the character and services of Miss Carolina Crook, who had served the Society and the Board for thirty-eight years at the time of her death on June 30.

Accrediting Technical Institutes

Upon inquiry, received from the Engineers' Council for Professional Development relative to the current expansion of Technical Institutes, the Board approved the proposal that E.C.P.D. undertake the program of accrediting such technical institutes.

Collective Bargaining

Communications relative to collective bargaining were presented from a number of sources with comments both favorable and unfavorable, respecting the Board's previous conclusions. In particular a considerable number of petitions calling for a mail referendum to corporate members was considered. After a full discussion, it was voted that this matter be not submitted to such a referendum, and that a communication be addressed to those who had signed the petitions telling of important factors which led the Board to its conclusion. It was reported that the constitutional amendments of thirty Local Sections had been approved.

Staff Activities

Reports of work by the staff in connection with engineering employment, engineers' salaries, federal legislation, and collection of data on proposed postwar construction were received.

Commendation of Student Chapters

In appreciation of the notable work carried on in the past months by a number of Student Chapters, special vote of commendation was given to 12 chapters. The list of Chapters thus honored is as follows: Iowa State College, Pennsylvania State College, University of Wisconsin, Manhattan College, Newark College of Engineering, Northeastern University, Tulane University, Virginia Military Institute, West Virginia University, Kansas State College, University of California, and Stanford University.

Cancellation of Dues

In accord with previously approved practice during war years, the dues of 293 members in the Armed Services were reported as cancelled since the previous meeting of the Board. Newly included in those eligible for cancellation, by vote of the Board, are members of the U.S. Public Health Service detailed to duty with the Army, Navy, or Coast Guard.

Postwar Planning

Report was received from the Chairman of the Committee on Postwar Construction and approved.

Technical Division—Committee Personnel and Policy

Responsibility for the designation of the personnel of Division committees was vested in the Executive Committees of the Divisions subject to the following policy: "The Technical Division Executive Committees, in selecting personnel of Division Committees, should select members by reason of their abilities and specific knowledge and should not bar any member of the Society from committee membership by reason of his employment or business connection."

Proposed Retirement System of U.E.T., Inc.

The Board approved the proposed retirement system for the employees of United Engineering Trustees, the Engineering Societies Library, and Engineering Foundation.

Salary Schedules

Extensive consideration was given to report of the Committee on Salaries, consisting of a revised Classification and Compensation Plan for Professional Civil Engineering Positions. Following minor changes, the report and schedule of salaries was approved for promulgation and publication, as shown in full on another page of this issue.

Other Matters

Various subjects were introduced and discussed, including financial matters, reports of standing committees, membership admissions and transfers and similar subjects; with appropriate action taken in all cases.

Cleveland Hospitality

Resolution was adopted expressing thanks and appreciation for various courtesies to the Board extended by the Cleveland Section and its members.

Adjournment

The Board ajourned to meet October 9, 1944, at Denver, Colo.

Extensive Questionnaire on Postwar Planning

THE Federal Works Agency began in mid-July a mailing of questionnaires to all states and counties, and all cities over 10,000 population, requesting that they report by July 29 the status of all postwar public works plans. State and Federal-aid highway projects were to be listed separately through U. S. Public Roads Administration reports from state highway departments and large cities.

The national survey was requested by the Special Committee of Postwar Economic Policy and Planning of the House of Representatives. Rep. W. M. Colmer (D.-Miss.), chairman, in a letter to Maj. Gen. Philip B. Fleming, Federal Works Administrator, urged "a complete report at the earliest possible date." Results of the questionnaires are to be tabulated by the Census Bureau on standard data cards before Congress reconvenes in September.

The questionnaires follow the general pattern introduced by the Society's Committee on Postwar Construction in its information program begun several months ago. Projects are to be classified by number and dollar volume in four stages of plan preparation: those completed to a point where bids could be let in 60 days; those in the design stage; those in the preliminary stage of preparation; and those on which no basic engineering studies have yet been made.

Land and construction costs are to be listed or estimated separately for the various types of projects which are grouped into: roads and streets, bridges and grade separations, airports and landing strips, sewer-water-sanitation, schools, hospitals, other public buildings, parks and recreation facilities, and miscellaneous. Further data are requested on local or state funds reserved for such projects, and other information on local ability to finance the work.

The chairman of the Society's committee urges cooperation of members in completing the reports as quickly as possible and returning them to the Federal Works Agency.

Status of Civil Engineering Construction Planning

CUMULATIVE data on postwar plans underway or completed, for the eighteen-month period ending June 30, 1944, show a total volume of $3,259,510,000. There was an increase in the past thirty days of approximately 23%, but over half of the increase was due to reported Federal earthwork projects in the State of California amounting to $331,000,000.

At the present time there is only $111,870,000 reported for private construction, or 3.4% of all engineering projects. Normally about 48% of such works should be privately constructed.

A study of the June 30, 1944, data by the Research and Development Division of the Society's Committee on Postwar Construction is summarized in the appended tables. The Committee is indebted to the McGraw-Hill Company for courtesy in furnishing the statistics.

TABLE I. VOLUME OF PLANNING BY PROJECT TYPE

TYPE	MILLIONS OF DOLLARS	PERCENTAGE OF TOTAL
Streets and roads	749.6	23.0
Earthwork, irrigation, etc.	589.7	18.1
Public buildings	523.9	16.0
Unclassified—public	401.9	12.3
Bridges	334.7	10.3
Sewerage	318.2	9.8
Water works	229.7	7.0
Commercial buildings	80.8	2.5
Industrial buildings	22.9	0.7
Unclassified—private	8.1	0.3
Total	$3,259.5	100.0

The total in Table I includes $539.5 millions of Federal Government projects, or about 16.5%. Streets, roads, and bridges comprise one-third of the total volume.

TABLE II. PLANS FOR WATER WORKS

LEADING STATES	THOUSANDS OF DOLLARS	PERCENTAGE OF TOTAL
New York	111,307	48.4
Texas	35,561	15.5
Massachusetts	19,079	8.3
Ohio	11,834	5.1
Washington	9,238	4.0
Maryland	8,300	3.6
District of Columbia	5,405	2.4
Nebraska	5,338	2.3
All others	13,614	10.4
Total	229,676	100.0

Table II shows that almost 90% of the volume of water-works projects orginates in eight states.

TABLE III. SEWERAGE PROGRAM

LEADING STATES	THOUSANDS OF DOLLARS	PERCENTAGE OF TOTAL
Ohio	60,773	19.1
New York	50,624	15.9
Illinois	31,195	9.8
California	23,155	7.3
Tennessee	18,656	5.8
Florida	15,054	4.7
Massachusetts	14,416	4.5
Maryland	14,258	4.5
Oregon	14,197	4.5
Indiana	13,606	4.3
Wisconsin	8,844	2.8
Washington	8,664	2.7
All others	44,735	14.1
Total	318,177	100.0

Approximately 86% of the sewerage program is reported from twelve states. Sanitation projects of Tables II and III represent 16.8% of the volume of plans in the design or completed stage.

TABLE IV. LEADING STATES IN ALL ENGINEERING CONSTRUCTION PLANNING

STATES	THOUSANDS OF DOLLARS	PERCENTAGE OF TOTAL
New York	781,194	24.0
California	464,314	14.2
Ohio	385,760	11.8
Texas	295,149	9.1
Washington	111,228	3.4
Pennsylvania	106,134	3.3
New Jersey	96,602	3.0
Michigan	96,067	2.9
Maryland	90,997	2.8
Indiana	86,586	2.6
Connecticut	79,990	2.5
Oregon	65,770	2.0
All others	599,719	18.4
Total	3,259,510	100.0

At present the major part of reported civil engineering construction is in twelve states and represents about 82% of the national volume.

Papers Available in Engineering Societies Library

IN THE Engineering Societies Library a file of technical papers has been established. These papers, which for one reason or another have not been published in Society publications, present data on a variety of subjects of interest to civil engineers. The manuscripts are available for reference at the Library or, if desired, photo-print copies can be secured by arrangement with the Library. Inquiries should be addressed to the Engineering Societies Library, 29 West 39th Street, New York, N.Y. Following is a list of papers recently filed:

SEDIMENTATION PROBLEMS

STAFFORD C. HAPP, "The Sedimentation Problem in the Middle Rio Grande Valley" (44 pages of text, plus tables, charts, and diagrams). Based on studies carried on for the U.S. Soil Conservation Service, this paper gives a factual report on sources and deposits of sediment carried by the Rio Grande River.

Pile Tests

L. T. Evans, Assoc. M. Am. Soc. C.E., "Sepulveda Dam Pile Tests" (23 pages of text, tables, charts, diagrams, and photographs). A complete report of testing methods and results of tests on various types of piling at the site of Sepulveda Dam near Los Angeles, Calif.

Cost Estimating on Earth-Fill Dams

Edward G. Smith, Jun. Am. Soc. C.E., "Empirical Formula for Estimating Cost of Flood Control Earth-Fill Dams" (17 pages of text, tables, charts, and diagrams). This report presents the derivation of a cost formula, a description of its use, and data illustrating its use.

Moments in Rigid Frames

Sheldon A. Keast, M. Am. Soc. C.E., "Simplified Methods for Computing Moments in Rigid Frame Buildings" (22 pages of text plus diagrams). The method explained in this paper avoids the use of empirical forms and bases its analysis on theoretical behavior of structural elements.

Rainfall and Runoff Characteristics

J. P. Bonner and W. B. Watson, "Intensity-Duration-Frequency of Precipitation in the Pacific Northwest as Modified by Snow Melt" (28 pages of text, bibliography, tables, and diagrams). This report covers an investigation made primarily to provide information for the estimation of runoff from streams in Washington, Idaho, and Oregon.

Analysis of Force Diagrams

Prescott Crout, "An Extension to the Theory of Reciprocal Force Diagrams for Plane Frameworks" (27 pages of text plus diagrams). This paper develops the fundamental theory of the use of force diagrams and demonstrates their use with systems which are redundant.

Tunnel Surveying

R. C. Thaxton, M. Am. Soc. C.E., "Charts and Diagrams for Tunnel Surveying at Boulder Dam" (11 pages of text, charts, diagrams, and photographs). Unusual sizes and shapes of tunnels at Boulder Dam made necessary unusual surveying methods which this paper describes.

Flood Control

A. L. Sonderegger, M. Am. Soc. C.E., "Influence of Physiographic Conditions on Flood Control" (31 pages of text, tables, and maps). This paper discusses the effects of disintegration of rock material, of erosion, debris transportation, and deposition on flood flow.

Death of Carolina Crook

Many members will learn with sorrow of the death of Carolina Crook, which occurred on June 30. Since an automobile accident in Mexico following the Los Angeles Convention a year ago, she had been in impaired health, but continued her work at Headquarters with her usual skill until the middle of February. By that time her duties following the Annual Meeting of the Board and the Society were completed. As would be expected, she left no "unfinished business."

Among those in active service of the Society, she had the longest term—38 years continuously. She came to the Society as a mere schoolgirl and advanced through application and ability until she held the most prominent position among its women employees.

To those visiting Headquarters she was known as confidential assistant to the Secretary. She met callers, handled routine correspondence, and in general had charge of the clerical work immediately connected with the Secretary's office and that relating to the Executive Committee and the Board of Direction, as distinct from the functions of the other offices at Headquarters. To this work she brought a vast amount of knowledge of Society affairs and procedures, an intimate acquaintance with many members, and a passion for both accuracy and completeness.

She attended all meetings of the Board as its recording secretary. In this position she not only came into contact with the officers of the Society, but with a wide group of Local Section officers and other Society members. Her poise and quiet efficiency made her justly popular. Few men, even among the officers and Headquarters personnel, had a wider acquaintanceship among engineers. Her loss is therefore a very real one.

Miss Carolina Crook

Many of the older members of the staff came to pay their last respects at the requiem mass held on Saturday, July 1, at the Church of the Blessed Sacrament, New York City. Notwithstanding insufficient warning, several former Board members were present, too. Interment was in Reading, Pa., where her parents also are buried. She leaves two sisters—Miss Mary Crook, with whom she lived in New York City, and Mrs. John Capebianco of Pittsburgh.

For her loyalty to everything connected with the Society, she will be mourned by many members. For her joyous temperament and her enthusiasm for life, she will long be remembered by her multitude of friends. To her the Society was more than an employer or a livelihood—it was an ideal, a life. She left it the richer for her long period of service in it.

Maintaining Society's Wartime Address List

A review of just one activity at Society Headquarters gives an idea of the complications arising from war conditions. Reference is made to the operation of the addressograph, and accompanying clerical routines, by means of which an up-to-date list of all members for use in distributing publications, notices and similar details is maintained.

The task of correcting this mailing list has been somewhat overwhelming during recent months. Starting in September 1943, with the mailing of the War Activities survey postcard (incidentally this resulted in the final receipt of 13,462 replies), there has been little or no relief in the bulk of details involved in the handling of the addressograph stencil work.

In 1943, the additions of new members totaled 1,576, the largest in the Society's history. Another complication was that the Post Office Department adopted the postal delivery zone system in many of the larger cities, requiring the addition of a zone number to a large percentage of the stencils. The volume of correspondence containing subsequent changes for listing in the Yearbook, following the return of the War Activities survey card, further increased the work in that department.

The constant changing of addresses and military titles of members in the Armed Forces added to the abnormal flow of civilian changes. To make matters worse, we lost the services of the two employees trained in the detailed work of maintaining the card records controlling the stencil changes. The present addressograph operators were all engaged within the past year.

It is interesting to note that thirteen separate and distinct detailed operations are required with every change of address, from the time it is received up to the point where the stencil is filed in its proper geographical position. A further matter of interest is the record of the number of membership stencils involved (changes only), excluding new members or other subscription files, Technical Division classifications, and so forth. The following is a comparison of the work of the past six months with that in the same period a year ago:

Year Ago	
Date	Stencil Changes
1942 Dec.	789
1943 Jan.	1,327
Feb.	735
March	431
April	564
May	878
Total	4,724

This Year	
Date	Stencil Changes
1943 Dec.	1,405
1944 Jan.	1,444
Feb.	1,320
March	1,475
April	1,416
May	1,057
Total	8,117

It was impossible to maintain a normal schedule of operation, and even with the addition of one extra person, there has been a delay of a month, and in some cases longer, before the new address could be incorporated in the records and address lists. Much overtime work has been necessary but by June 1, to the relief of all concerned, this bottleneck was entirely eliminated.

It should perhaps be mentioned that the filing of stencils can be done only at certain times of the month—when the containers are not being addressed for CIVIL ENGINEERING or PROCEEDINGS. When the stencils are in actual use for such purposes they are not accessible for the making of changes.

No attempt has been made here to describe the routines involved in connection with the addition of new members, mail withheld, resumption of publication service upon payment of arrears, Technical Division changes and additions, special subscriptions for publications, and similar file and stencil changes. These are normal Headquarters operations which go forward in any circumstances. The accomplishment of the staff was in overcoming the handicaps of the extra work caused by the war without neglecting regular duties.

Appointments of Society Representatives

CHARLES B. BURDICK, M. Am. Soc. C.E., will continue to serve as the Society's representative on the Washington Award Commission.

J. K. FINCH, M. Am. Soc. C.E., has accepted reappointment as one of the representatives of the Society on the Board of the Engineering Societies Library for the three-year term, October 1944 to October 1947.

S. C. HOLLISTER, M. Am. Soc. C.E., has accepted an appointment as one of the Society's representatives on the Engineers' Council for Professional Development for the three-year term, October 1944 to October 1947.

News of Local Sections

Recent Activities

CENTRAL OHIO SECTION

A number of A.S.T.P. students in civil engineering at Ohio State University were guests of the Section at the May meeting, which took place on the campus of the university on the 24th. The principal speakers were Maj. A. R. Morrison, who discussed the objectives of the Army Specialized Training Program, and Director Frank C. Tolles, who reported on recent activities of the Society. During the evening it was announced that the civil engineering student graduating from Ohio State University with the highest scholastic standing during the past year is Don P. Schiesswohl. Since Mr. Schiesswohl is already a Junior in the Society, it was decided to send him a cash prize.

COLORADO SECTION

On June 12 members of the Section heard Lt. Col. Carl Stenersen, Norwegian military attaché to the 10th Light Infantry of the U.S. Army, speak from first-hand experience on the infiltration of fascism into the social, economic, and political structure of the German nation. Colonel Stenersen, who was formerly a military observer in the German army, also described the fifth column methods that led to the downfall of Norway in 1940.

DISTRICT OF COLUMBIA SECTION

"Water Supply Development in the Maryland Suburbs of Washington" was the subject of discussion at the June meeting of the Section, the principal speaker being Harry R. Hall. Mr. Hall, who is chief engineer of the Washington Suburban Sanitary Commission, called attention to the tremendous growth of these suburbs in recent years and showed pictures of the impounding dam and reservoir works. Of special interest was his description of the filtration plants, which exemplify an unusual design. The rest of the meeting was given over to committee reports and to a few remarks by E. L. Chandler, the Society's representative in Washington.

FLORIDA SECTION

Members of the Florida Section enjoyed two dinner meetings in May. On the 19th, Col. H. D. Mendenhall discussed high lights of the St. Louis Meeting, and various members commented on aspects of his résumé that were of interest to the Section. Then Winston Wheat spoke on postwar construction and recommended that legislation setting up a tax for such construction be enacted. There was a special meeting on the 30th in honor of Malcolm Pirnie, President of the Society, who addressed the group on the postwar plans of the Society. He also commented briefly on the action of the Board of Direction in recommending the formation of collective bargaining committees.

GEORGIA SECTION

At the regular June meeting of the Section Colonel Banton gave an interesting account of the invasion of North Africa. During the business session it was announced that this year the Section's annual prizes of Junior membership in the Society go to Jacque W. Cannon, president of the Student Chapter at the Georgia School of Technology, and Ped F. Jarvis, Jr., of the same institution. A special meeting was called on the 26th to discuss postwar construction work and to make plans for a two-day fall meeting.

INDIANA SECTION

At a dinner meeting of the Section, held in Hammond on May 24, H. G. Woodbury, Colonel, Corps of Engineers, U. S. Army, discussed equipment and construction methods used by the Army engineers in the present war. Colonel Woodbury, who is stationed in Chicago, emphasized the fact that engineers on the home front should give serious consideration to postwar problems of the profession. During the evening the Section's prizes of Junior membership in the Society were presented to Safford McMyler, of Purdue University; William R. Colclesser, of Rose Polytechnic Institute; and Michael T. McGlew, of Notre Dame.

LOUISIANA SECTION

Numerous business matters were discussed at the June meeting of the Louisiana Section, which took place in New Orleans on the 5th. Several committee reports were also heard, after which there was a talk on the work of the Seabees. This was given by W. R. MacGregor, lieutenant, Civil Engineering Corps, U.S. Naval Reserve. The showing of a Navy film, entitled "Building Advance Naval Bases with the Seabees," concluded the program.

MID-MISSOURI SECTION

On June 27 members of the Section met in Rolla to hear T. P. Pendleton, chief topographic engineer for the U.S. Geological Survey, speak on postwar mapping. Mr. Pendleton traced the development of map making from the crude sketches by artists to the present use of aerial photographs and multiplex machines. The explanation of highly technical developments was aided by lantern slides. Mr. Pendleton emphasized the importance of good maps for general public use, as well as for engineers. It was a shock to his audience to learn that less than half the area of the United States is covered by up-to-date maps.

NEW MEXICO SECTION

The technical program at the May 31 meeting of the Section consisted of a talk on "Suspended Sediment in the Colorado River"—given by C. S. Howard. Mr. Howard, who is senior chemist for the U.S. Geological Survey, demonstrated several types of silt samplers and explained their use. The lecture was illustrated with slides showing graphs of the relation of silt to runoff, comparisons of various stations, and wet and dry cycles.

North Carolina Section

The spring meeting of the North Carolina Section, consisting of afternoon and evening sessions and a dinner, took place in Greensboro on May 26. The first speaker on the afternoon program was Henry A. Yancey, city manager of Greensboro, whose subject was "Municipal Government in the Postwar Period." In the unavoidable absence of T. S. Johnson, chairman of the local Committee on Employment Conditions, N. P. Hayes discussed the findings of the committee. Following dinner in the evening, J. N. Todd presented an unusual paper on "Track Scales with Particular Reference to Locomotive Weighing." Such scales, he said, have shown discrepancies of upwards of 10,000-lb axle loads between design loads and actual occurrence. Mr. Todd is superintendent of scales and work equipment for the Southern Railway System. The other speaker on the program was T. R. Higgins, chief engineer for the American Institute of Steel Construction, whose subject was "Emergency Specifications and Postwar Engineering."

Oregon Section

An illustrated talk on the proposed flood-control projects in the Sacramento and Joaquin River basins comprised the technical program at the May meeting of the Oregon Section. This was given by Frank Kochis, principal engineer for the U.S. Engineer Department at Sacramento, Calif. During the evening a letter from President Pirnie, commenting on the Society's program for postwar construction, was read.

Panama Section

The Panama Section reports that on April 3 Thomas Maddock, Jr., chief of party for the Food Supply Division of the Institute of Inter-American Affairs, discussed the work of the Institute and, particularly, the program now under way in the Republic of Panama. At the May meeting—held in Balboa on the 1st—the speaker was H. F. Finch, chief of the Materials Control Section of the Supply Department of the Panama Canal. Mr. Finch discussed the general functions and activities of the storehouses and oil-handling plants, including operations under the various regulations and prescribed procedures of the War Production Board. A film outlining the general purpose of the controlled materials plan illustrated Mr. Finch's remarks. The Panama Canal has been confronted with many problems and difficulties in the procurement of supplies during the past few years of expanded activity, and Mr. Finch's talk was therefore of unusual interest.

Sacramento Section

Following an annual custom, the Speakers' Club of the Section supplied the program for the May 9th meeting. James Gallagher explained the derivation and use of a formula for computing the field and design-room time required for preparation of postwar bridge plans. The latest procedure used by the state in correlating rainfall and runoff was described by Carl B. Meyer, while Earnest A. Bailey discussed a flood-control survey on the Emory River Basin in Tennessee. On May 23, Fred J. Grumm, assistant state highway engineer, repeated his recent address to the Los Angeles Section on postwar freeway projects as an essential part of functional city planning.

New techniques in aerial mapping were explained to the Section by Thomas P. Pendleton, member of the District of Columbia Section and chief of the topographic division of the U.S. Geological Survey, in an illustrated lecture on June 6. The meeting was scheduled for the evening in order to give time for discussion. By special invitation, members of the American Society of Photogrammetry and the Northern California Geological Society were present as guests.

San Francisco Section

The technical program for the regular bi-monthly dinner meeting, held on June 20, consisted of an address by James F. Rae, plastics consultant, on "Plastics and Their Possibilities in Postwar Developments in California." Mr. Rae described developments in the field of plastics to date and outlined the postwar uses of plastics, when present restrictions are lifted and they are made available to the public. On May 17, the members attended a special dinner meeting, at which the Navy film, "Seabees in Action," was shown.

Two talks were the feature of the May 25th meeting of the Junior Forum. These were given by Arthur A. Hoffman, engineer for the Golden Gate Iron Works, whose subject was "Problems of Conducting a Steel Fabricating Business," and J. D. Worthington, assistant engineer for the Pacific Gas and Electric Company, who discussed the design and construction of surge chambers. The topic for discussion was: "Should the Junior Civil Engineer Plan for Possible Foreign Service in the Immediate Postwar Period?"

Spokane Section

On May 12 members of the Spokane Section saw two films—"Mine to Metal" and "Fabrication of Aluminum"—through the courtesy of the Aluminum Company of America. The films were shown by Gordon James, of the Aluminum Company of America, who answered many questions concerning this metal. The June meeting took the form of an inspection trip to the Spokane Army Air Depot, which was made possible through the courtesy of Maj. E. S. Crockett. After a dinner and informal meeting at the Officers' Club the group boarded a bus, which transported them on the tour of inspection. The members viewed a large portable hangar, and were then conducted through the engine test building, the engine repair building, and the large repair hangars. Major Crockett explained the role of the Army Air Depot in the war and discussed the functions of the different units.

Tacoma Section

The June meeting of the Section took place at the Gray Goose Inn near Tacoma on the 20th. Following an excellent chicken dinner, W. J. Ryan reported on the activities of the Section's Postwar Planning Committee, and R. C. Knapp discussed the work of the Section's Committee on Employment Conditions. C. J. Rossio, of the Johns-Manville Corporation, then showed colored moving pictures—of the "Big Inch" and "Stone Canyon." The latter described construction work on the water supply line to and from the Stone Canyon Reservoir of the City of Los Angeles. Both films were furnished through the courtesy of the Barrett Division of the Allied Chemical and Dye Corporation.

Texas Section

The Fort Worth Branch of the Section held a luncheon meeting on June 12, at which C. M. Davis showed motion pictures of the construction of the Southern Pacific Railroad's bridge across the Pecos River. The piers were constructed by the sliding-form method and completed in what is believed to be record time. During the business session the following new officers were elected for the coming year: D. W. Robinson, president; Uel Stephens, vice-president; and Henry Rockwood, secretary.

West Virginia Section

"Wartime Problems of the Metallurgist" were discussed by W. A. Mudge at the May dinner meeting of the Section, which was held in Morgantown on the 12th. Dr. Mudge is assistant director of technical services for the International Nickel Company, and a motion picture showing the manufacture of nickel alloys concluded the technical program. During the business meeting C. W. Okey, representing the Society's Committee on Employment Conditions, spoke on collective bargaining for engineers. Mr. Okey is principal civil engineer for the Tennessee Valley Authority.

Wisconsin Section

Malcolm Pirnie, President of the Society, was the Section's guest of honor at a dinner, which was given in Milwaukee on June 14. After dinner Mr. Pirnie discussed the activities in which the Society is now engaged. He was followed by A. M. Rawn, Director of the Society, who spoke on the progress made by the various Sections in establishing collective bargaining units. After the dinner the Section held a joint meeting with the American Water Works Association, which was addressed by Col. William N. Carey on "Planning for Postwar Construction"; A. M. Rawn on "Labor Organizations in the Professional and Municipal Field"; and Edward Falck, director of the Office of War Utilities on "Program and Policies of the Office of War Utilities."

ITEMS OF INTEREST

About Engineers and Engineering

Attendance Record Set at War Production Conference

At a record-breaking meeting, 4,000 enthusiastic and interested engineers, scientists, and manufacturers gathered on March 30 at the Hotel Sherman in Chicago, for the Second Conference on War Production and related problems, sponsored by the Chicago Technical Societies Council. The clinic was held at the request of the War Production Board and was in cooperation with the Board and the Army and Navy. The purpose of the conference was to expedite the war effort by providing a means for the exchange of mutually beneficial ideas among engineering, technical, and scientific societies, and interested manufacturers in the Chicago area. The spirit of cooperation between manufacturing and technical staffs was ample evidence that a large group of people could get together and work for the common good. The conference was held in a series of 33 panel sessions in which all phases of war production and related problems were discussed. All panel discussions were led by recognized experts in their fields.

Composed of the representatives of 36 societies, including the American Society of Civil Engineers, the aims of the Chicago Technical Societies Council are: (1) to provide a medium for cooperative action by the engineering, technical, and scientific societies in the Mid-West on matters of mutual interest which are beyond the scope of individual societies or which can be performed better by cooperative action; (2) to provide means for more effective public service by the member societies; and (3) to cultivate greater appreciation by the public of the part which the engineering, technical, and scientific professions have contributed to human welfare.

Some Lumber Sizes Available

Specified sizes of seven species of American woods have just been listed by the War Production Board as "materials available in excess of current demands." Issue Number 12 of *Material Substitutions and Supply List*, which is published as a guide to the current relative availability of important materials, classifies commodities in three groups:

Group I. Materials which are insufficient for war plus essential industrial demands within the limits imposed by existing administrative controlling orders.

Group II. Materials of which there is at present a sufficient supply to meet war demands plus essential industrial demands within the limits of existing controlling orders.

Group III. Materials which, except for local shortages, are readily available for essential uses—materials which should be used wherever possible in place of those in Groups I and II.

Lumber which falls in Group III, as of April 15, was given as:

Western Red Cedar, timbers
Pecky Cypress, timbers
Douglas Fir, dimension (3 and 4-in. thicknesses only), timbers
Western Hemlock, dimension (3 and 4-in. thicknesses only), timbers
Western Larch, dimension (3 and 4-in. thicknesses only), timbers
Southern Pine, dimension (3 and 4 in. thicknesses only), timbers
Redwood, timbers.

Asphalt Conference Papers Issued

The Asphalt Institute announces that the Proceedings of the First Illinois State Asphalt Conference have been issued, but at present are not available for very general distribution except in Illinois. However, reprint pamphlets of two of the papers are generally available upon request to any office of the Institute. The executive office is located at 801 Second Avenue, New York 17, N.Y.

The papers available are "Construction and Resurfacing of Primary Highways" (Construction Series No. 69) by Bernard E. Gray, and "Soil Bearing Tests and Flexible Pavement Design" (Research Series No. 10) by Prevost Hubbard.

Steel Industry to Study Reinforcing Steel

Basic study of the uses of steel bars for concrete reinforcement is being inaugurated by a Committee on Reinforced Concrete Research recently organized by the American Iron and Steel Institute. The membership of the committee includes representatives of 21 producers of both new billet and rail-steel reinforcing bars for concrete. To plan and supervise the investigations, R. R. Zipprodt, M. Am. Soc. C.E., has been employed as research and consulting engineer. Mr. Zipprodt graduated as a structural engineer from the University of Illinois in 1915, and has had long experience in concrete research and design.

The Committee has outlined an initial program of work covering a minimum of three years. In general, it is planned to study the extent of any gaps which may exist in the technical data on which design regulations and specifications are customarily based. It is then proposed to initiate programs of research to develop new test data or to supplement existing data. Several research projects have already been authorized.

Work is already under way on the first, a comprehensive investigation of problems relating to the proper design of square and rectangular footings. No research on footing design has been reported since 1913. Since then many major improvements have been made in the materials utilized in reinforced concrete construction.

A second project will involve the investigation of unit stresses permissible in the design of reinforced concrete slabs and beams. The study aims to determine the extent to which the ratio of slab thickness to span affects the unit stresses in the slab and consequently its deflection. The extent to which the span of slabs or beams should be limited by the size or type of reinforcement will also be investigated. A collateral phase of this project will be a study of the effect of plastic flow, particularly on structural members subject to their own dead weight as well as to uniformly distributed live loads.

A third project will cover the classification of all types of deformed bars which may be developed as a result of the purpose of the bar industry to develop a standard or universal type of deformed bar, for concrete reinforcing. Such classification tests are to be conducted in parallel at three well-recognized research laboratories after the standard form of test for bond now in process of development by the American Concrete Institute (Committee 208, Bond Stress) has been approved.

Why Engineers Get Gray

Unconscious humor occurs now and then in the dry pages of the Congressional Record and in records of committee hearings, as witness the following dialogue. The dramatis personnae are Congressman John Taber of New York and Dr. H. W. Bashore, Commissioner of Reclamation, U.S. Department of the Interior. The scene: Appropriations Committee hearing room. The subject: A submerged weir on the Colorado River as proposed by the Reclamation Department. Extracts are quoted from the official report of hearings on the "First Deficiency Appropriation Bill for 1944."

"Mr. Bashore. . . . . Of course, the river slopes, and the weir would probably be about 750 ft long . . . ."

* * * *

"Mr. Taber. How high does the river have to be raised in order to do this job?"

"Mr. Bashore. It would have to be raised about 6 ft . . . ."

"Mr. Taber. It seems to me that your estimate for this is pretty high. Why would it take so much money? It does not have to be a very wide base, does it or instance, you have 600 ft by 6 ft, and hat is 3,600 sq ft, or 400 sq yd.

“It does not cost more than $10 a yard. it is 2 yd wide that would be about as ide as you need to have it. That would e 1,600 cu yd of stone, which does not ost over $10 a yard to get it out and uarry it and get it ready to put in.”

“Mr. Bashore. No, Sir.”

“Mr. Taber. Why would it cost so verlasting much? It is not a tough job? is not a cement job?”

Page Prof. N. G. Neare.

Report on Hydrologic Data Available

The Los Angeles County Flood Control istrict has issued an Annual Report on ydrologic Data for the Season of 1941–942. This report—the twelfth of a series annual or biennial reports—includes ata on precipitation, evaporation, runoff, am operation, ground water, and con-rvation.

The collection, compilation, and publi-ation of this type of data is of ever-creasing value to those interested in ydrologic study, planning, and the de-gn and operation of flood-control and nservation projects.

Need for Civil Engineers in Industry Indicated

In a recent survey conducted by the War anpower Commission it was found that substantial shortage of civil engineers ists in certain civilian occupations. The search indicated that between January and July 1, 1944, 3.3% more civil engi-eers would be added to payrolls of in-ustry if they were found available. It as estimated that 68% of the needed rsonnel would be drawn from the pool new college graduates.

While the report showed a need for 3.3% ore men in civil engineering fields, it owed a need for 21% more men in radio gineering, 17.1% in petroleum engineer-g, 15.9% in aeronautical engineering, 1.3% in chemical and mechanical engi-eering, and 7.9% in electrical engineering. n that the survey was made of “indus-ial personnel,” no indication of the eeds in other civil engineering fields was iven.

N. G. Neare's Column

Conducted by

R. Robinson Rowe

The August meeting of our Engineers Club attracted a few armchair strategists ho hoped Professor Neare would give way the location of our secret radio sta-ion in an isolated part of a Jap-held island. Sorry,” he said, “but the OWI has re-eased nothing—not even that it's less han 0.02 sec by radio from Tokyo. So e have only the problem of the mast uys; if three of these were 350, 450, and 500 ft long, how long was the fourth?”

“Since the guys were anchored to the corners of a square, I found a remarkable relation between their lengths,” declared Joe Kerr. “If lengths of guys in clockwise order are a, b, c, and x, then $a^2 + c^2 = b^2 + x^2$, regardless of the height of the mast. So, ignoring all that extraneous data, $x = \sqrt{350^2 - 450^2 + 550^2} = 471.7$ ft.”

“Maybe so,” hedged Guy Wyer, “but I tried to fit Joe's answer to the extraneous data and found that the 6 guys, including the leaders down the mast, required 2,221.7 ft of wire, which wouldn't girdle the 565.1-ft square by 38.6 ft. Since the order of the guys wasn't specified, I tried the unknown guy opposite the 350-ft one and found $x = 618.5$; then I had 2,368.5 ft of wire which didn't girdle the 610.7-ft square by 74.4 ft. The third combination, with the unknown guy opposite the 550-ft one, gives $x = 150$; this won't reach the ground. So the extraneous data are inconsistent.”

“Not at all,” countered Ken Bridgewater. “The Seabee topped the 200-ft tree with a spider, increasing the height of the mast. For the first combination, the spider must be stubbed up 7.5 ft; then there will be 2,236 ft of wire to girdle a 559-ft square. For the second, a 14.4-ft stub makes 2,397 ft of wire to girdle a 599.3-ft square.”

“I wouldn't overlook that third combination,” suggested Cal Klater. “Webster is very liberal with the verb ‘to top’ with the ambiguous meanings ‘to cap,’ ‘to put a top on,’ and ‘to crop.’ Cropping the tree to a 112-ft height lets the 150-ft guy reach the corner of a 431-ft square, which will just be girdled by the 1,724 ft of wire.”

“I'll bet on Cal's answer,” exclaimed Amos Keatow. “I've essayed in the forest when the loggers have trimmed a spar tree and the *topping* is a spectacular feat. That's the word for cutting off the top of the tree where it's strong enuf to be guyed for a gin pole.”

“So it is,” agreed the Professor, “and 150 ft is the answer. Joe's formula will give three answers in three minutes, but picking the right one depends on knowing that a tree is topped down and not up.

“Two of the Seabees are Lieutenants Heald and Nobmann, whose off-duty hours are spent at cribbage. In no time the gang dubbed them Heels and Nobs, respectively, and suggested that Heald should always peg His Heels and Nobmann His Nobs. Heald demurred until they worked out a system of scoring that fits the laws of averages and probabilities. Now, no matter who holds His Nobs, Nobmann pegs one; no matter who cuts His Heels, Heald pegs a number that seems to depend on the bottom card of the deck. How could that be?”

[*O'Kay (Otto H. S. Koch) was both Klater and Bridgewater. Others either ignored the girdling requirement or generously branded it a typographical error. The new problem developed in correspondence with Mason D. Pratt, who makes a hobby of the probabilities in cribbage.*]

NEWS OF ENGINEERS

Personal Items About Society Members

Gibb Gilchrist was recently elected president of the Agricultural and Mechanical College of Texas. Since 1937 he has been dean of engineering at the college.

Otto Holden, chief hydraulic engineer for the Hydroelectric Power Commission of Ontario, is being awarded the honorary degree of doctor of engineering by his alma mater, the University of Toronto, at its annual commencement exercises. Mr. Holden has just completed a year of office as president of the Royal Canadian Institute.

N. M. Newmark, research professor of civil engineering at the University of Illinois, has been granted a six-month leave of absence to undertake a special overseas assignment as a technical observer and expert consultant to the Army for the Office of Field Service.

John B. Wilbur has been appointed acting head of the department of civil and sanitary engineering at the Massachusetts Institute of Technology, succeeding **Theodore B. Parker**, who died in April. Previously he was professor of structural engineering at the Institute.

Stanley R. Biesack has been promoted from the rank of first lieutenant in the Corps of Engineers, U.S. Army, to that of captain. He is post engineer officer at the Boston Port of Embarkation. Prior to entering active military service, Captain Biesack was junior structural engineer for the TVA at Chattanooga, Tenn.

Eric Haquinius recently returned to Mineral Wells, Tex., where he will resume his consulting work after a year and a half overseas as a lieutenant colonel in the Corps of Engineers, U.S. Army. His work overseas was as port engineer for a large and important Tunisian port.

Chilton A. Wright has been given a leave of absence from his position as professor of hydraulic and sanitary engineering at the Polytechnic Institute of Brooklyn in order to join the staff of the Experimental Towing Tank at Stevens Institute of Technology, where he will be engaged on various confidential research projects for the Navy's war-expansion program.

William R. Seeger was recently commissioned a first lieutenant in the Sanitary Corps of the U.S. Army, attached to the U.S. Public Health Service. Until lately he was an industrial engineer for Montgomery Ward at Oakland, Calif.

Robert B. Rothschild, Jr., is now a partner in the E. T. Haas Company, of San Francisco, Calif. For some years previously he was principal engineer for MacDonald and Kahn, Inc., of the same city.

Howard L. Kegler, formerly senior structural engineer in the California State Department of Public Works, at San Francisco, has been appointed chief of the engineering division of the San Francisco district of the Corps of Engineers, U.S. Army.

Laurence T. Gaylord, vice-president of the Atlantic, Gulf, and Pacific Company, of New York City, has been awarded the Marston Medal for 1944 for his ". . . sustained efforts . . . in the advancement of the art of hydraulic dredging, in providing better harbor facilities, better waterways, and recently roadways and airports for the United States. . . ." The award, which was established by Iowa State College in 1938 through the generosity of Anson Marston, Past-President and Honorary Member of the Society, is given annually to an engineering alumnus of the college in recognition of engineering achievement. Mr. Gaylord has been with the Atlantic, Gulf, and Pacific Company since his graduation in 1904.

Thomas R. Archibald was recently promoted from the rank of major in the Corps of Engineers, U.S. Army, to that of lieutenant colonel. At present he is stationed in New Guinea.

William S. Little is now a major in the Corps of Engineers, U.S. Army, the rank representing a promotion from that of captain. Major Little has been assigned to the Army Engineer Board at Fort Belvoir, Va.

Norman H. Davidson, major, Corps of Engineers, U.S. Army, has been transferred from Syracuse, N.Y., where he was post engineer at the Army Air Base, to the Engineer Office of the 2d Corps Area in New York City.

Arthur B. Mason has established a private civil engineering practice at San Benito, Tex. Until lately he was associate engineer for the International Boundary Commission, with headquarters in San Benito.

Henry D. Dewell, consulting engineer of San Francisco, Calif., announces that his son, **Robert D. Dewell**, will be associated with him under the firm name of Dewell and Dewell, with offices at 604 Mission Street, San Francisco 5. The latter was formerly project manager for the William P. Neil Company, of Hawthorne, Nev.

Robert H. Dodds has been promoted from the rank of lieutenant (jg) in the Civil Engineer Corps of the U.S. Naval Reserve to that of lieutenant. At present he is stationed at Dover, N.J.

Walter Emmons, associate professor of civil engineering at the University of Michigan, was recently appointed assistant dean of the college of engineering.

James H. Stratton is now a brigadier general in the Corps of Engineers, U.S. Army, the rank representing a recent promotion from that of colonel. General Stratton is chief of the supplies division in the European Theater of War.

Glenton G. Sykes, formerly assistant area engineer for the U.S. Engineer Department engaged on military construction, has been appointed city engineer of Tucson, Ariz.

William M. Caldwell has been appointed assistant to the executive vice-president of the Gray Iron Founders' Society (national association of manufacturers of engineering gray irons), with headquarters in the Washington (D.C.) office of the organization. Until lately Mr. Caldwell was senior business specialist in the iron and steel section of the Office of Price Administration.

Edward P. Lupfer, consulting engineer of Buffalo, N.Y., was recently honored by his alma mater, the University of Kansas, with a "citation of distinguished service" for his "signal contributions to the utilities, the industries, and the beauty of Buffalo and the Niagara Frontier." The citation, which takes the place of honorary degrees in Kansas, was presented to Mr. Lupfer, at the commencement exercises held on June 29. Mr. Lupfer is president of the Edward P. Lupfer Corporation, which he founded in 1923. He served as Director of the Society from 1930 to 1934, and as Vice-President in 1936 and 1937.

Russell C. Brinker, lieutenant commander, Civil Engineering Corps, U.S. Naval Reserve, is now on duty in the Pacific. Until recently he was public works officer at the Naval Advance Base Depot at Davisville, R.I.

George F. Nicholson has been promoted from the rank of commander in the Civil Engineering Corps of the U.S. Naval Reserve to that of captain. He is stationed in San Francisco, Calif., where he is in charge of naval construction work. Before being called to active duty Captain Nicholson was engineer for Los Angeles Harbor.

A. Stuart Collins, Buffalo (N.Y.) consultant, announces the formation of a partnership with his associate, John A. Beane, to continue the practice of engineering under the name of Collins and Beane, in collaboration with John Y. Sloan, architect. Their offices will be located at 232 Delaware Avenue, Buffalo 2, N.Y.

Robert E. Stiemke, formerly director of the North Carolina State College Engineering Experiment Station, has been commissioned a first lieutenant in the U.S. Army and assigned to the U.S. Public Health Service as a sanitary engineer.

Aubrey E. Palmer has severed his connection with the engineering faculty at the University of Mississippi in order to accept a position as assistant professor of civil engineering at Duke University. He will fill the vacancy caused by the resignation of **Roy K. Jacobs**, who recently accepted a commission as lieutenant (jg) in the U.S. Naval Reserve.

L. D. Worsham, brigadier general, Corps of Engineers, U.S. Army, was recently awarded the Legion of Merit for "his outstanding services" as division engineer of the Great Lakes Division at Chicago, Ill., from May 1942 to April 1943. At present General Worsham is assistant chief of engineers in charge of troops, with headquarters in Washington, D.C.

Salvatore Grasso is now a first lieutenant in the Sanitary Corps of the U.S. Army, assigned to the U.S. Public Health Service. Prior to being commissioned he was an instructor in civil engineering at the College of the City of New York.

Paul Andersen, previously professor of structural engineering at the University Minnesota, now has the rank of majo in the Corps of Engineers, U.S. Army He is stationed at the Supreme Headquarters of the Allied Expeditionary Force

Robert B. Brooks, Jr., was recently promoted from the rank of major in th Coast Artillery Corps of the U.S. Army to that of lieutenant colonel. Formerly a engineer with the Shell Oil Company Colonel Brooks was called into service i January 1941 and served at various post until ordered to Britain a year ago.

DECEASED

George Lewis Bean (M. '12) o Philadelphia, Pa., died recently. Early in his career (1901 to 1906) Mr. Bean was with the American Pipe and Construction Company, of Philadelphia; from 1906 to 1908, with Whitmer and Cocco, Philadelphia firm of engineers and contractors; and from 1909 to 1912, with the National Board of Fire Underwriters in New York. Later he was with the Washington County (Maryland) Water Company and, during the first World War, was supervising engineer on cantonment construction. From 1919 until 1940 he was in private practice in Philadelphia.

Thomas Byrnes Brogan (Assoc. M. '10) civil engineer for the Highways and Sewers Department of the Borough of Brooklyn, N.Y., died on June 11, 1944. Mr. Brogan, who was 64, had been in the Bureau of Highways and Sewers, Office of the Borough President of Brooklyn, since 1913. Earlier in his career (1905 to 1909) he was engineer-inspector on the construction of the Pennsylvania Railroad tunnels and terminal in New York, and from 1909 to 1913 was assistant engineer on the New York City Board of Water Supply.

Robert Hall Craig (Assoc. M. '19) consulting engineer of Harrisburg, Pa., died in Philadelphia on June 18, 1944, at the age of 51. Mr. Craig had spent most of his career in consulting engineering work in New York and Harrisburg, specializing in the design of water works and sewage-treatment plants. Earlier he had been in the Surgeon General's Department in Washington, and designing engineer for the Construction Division of the Army.

Edward Jacob Dauner (M. '30) surveyor and regulator for the Bureau of Engineering Surveys and Zoning, 4th Survey District, Philadelphia, Pa., died in that city on March 20, 1944. He was 62. Mr. Dauner had been with the City of Philadelphia continuously since 1900—first as draftsman and, then, as principal assistant for the 9th Survey District. From 1919 to 1926 he was assistant engineer in charge of the General Plans Division, and since 1926 he had been surveyor and regulator.

Nathaniel Warren Green (M. '25) supervisor of the planning department, St. John's River Shipbuilding Company at Jacksonville, Fla., died there on May 24, 1944, at the age of 56. From 1910 to